1971-10

SUMMARY REPORT

DEWATERING SCHEME FOR OVERBURDEN

OF

GREAT CANADIAN OIL SANDS

FORT McMURRAY, ALBERTA

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CONTENTS

INTRODUCTION

The purpose of this report is to make recommendations regarding methods of dewatering of the overburden on the G.C.O.S. lease. I have made no attempt at any financial evaluation of the procedures. It was felt that this could not adequately be done without intimate knowledge of the necessary financial restraints. I have attempted to outline as clearly as possible the necessary equipment, construction, and operating procedures for the various dewatering options available so that a proper cost analysis can be made.

Some previous work was done by Alain Kahil (1968a, 1968b, 1969), a graduate student in Geology at the University of Alberta. I am in possession of his work and have reviewed it thoroughly. This report, however, is the result of my own analysis of the available data and I feel in no way committed to any previous recommendations, formal or informal, by the former worker.

Published references utilized include Luthin (1966), Glover (1966), Clark (1960), Linckens (1965), Thornthwaite (1957), and Walton (1962). I will refrain from extensive definitions of hydrogeologic terms and refer the reader to Todd (1959) and Davis and DeWiest (1966).

SUMMARY AND RECOMMENDATIONS

A plan for dewatering the overburden of G.C.O.S. lease #86 has been devised. This plan has several required portions; within some of these portions alternative methods for accomplishing the same goal are presented for cost analysis. I have also outlined the plan which I feel will best accomplish the dewatering irregardless of cost.

Hydraulic conductivities of materials constituting overburden at the lease have been measured to range between 4800 and 0.5 gpd/ft^2. It is felt that the aquifer tests upon which these figures are based are not entirely valid. It was judged that a more likely range was between 200 and 6 gpd/ft^2. Calculations are presented utilizing both these figures with the indication that actual values will likely be between the two extremes.

Drainage of the muskeg is necessary to reduce infiltration of precipitation into underlying units. This can be accomplished by ditches, moving water under gravity flow, and spaced about 800 feet apart. These ditches would be about 2 to 3 feet deep and could be constructed by a small dragline or by specialized equipment.

The existence and monitoring of an adequate piezometer network is essential to the success of the total plan. Piezometers can consist of 1 1/4 inch iron pipe with a short screened sandpoint. Five new piezometer locations must be established requiring about 450 feet of pipe and drill hole. Many existing piezometers are too shallow to adequately measure large drawdowns of the water table. Ten of these will require deepening necessitating about 800 feet of drill hole and pipe. Once-monthly monitoring of the piezometers is necessary, requiring about one man-day of labor.

It is implied in the rest of the discussion that the necessary appurtenances for removal of water from the confines of the lease to the Athabasca River are provided. This could take the form of shallow ditches or piping.

The emplacement of wells was deemed to be the best approach to dewatering of the area where operations will take place until about 1976. Piezometer observations will be utilized to indicate the actual necessity for installation of dewatering wells in this area. These wells should be installed two years before overburden removal is expected to reach a particular locale. Pumping should be done continuously year round. There are nine anticipated locations of these wells totaling about 460 feet in depth. These wells should be partially screened, randomly slotted, and gravel packed for most efficient production. Some equipment will be reusable since only about four or five wells will be in operation at any one time.

Dewatering of the portion of the lease in which operations will take place after 1976 involves two main divisions with cost-based alternatives possible within each of these.

The first division consists of operations designed to remove water from the overburden and to reduce movement of groundwater into the lease from the southwest. Basically it involves the construction of a hydraulic barrier within the lease and close to its southwestern edge. This can be accomplished in one of two ways:

1) A main ditch 14,500 feet long, averaging 60 feet deep. The grade of this ditch will be such as to cause water to flow to its north end at which point it can be discharged either by: a) pumps with a total capacity of about 1 to 5 mgd pumping year round, or b) an outlet ditch 3600 feet long that will transport water without pumping to the escarpment.

This ditch may be constructed in short increments of minimum length such that the end of the ditch is always at least 4 years ahead of overburden removal. This choice will negate the possibility of an outlet ditch and will require pumps of 1 mgd capacity to pump year round. It is also emphasized that this increment approach will lead to less efficient removal of overburden water and will have implications as explained later.

2) A row of about 40 wells spaced at about 300 feet along the trend of the ditch, averaging 65 feet deep. This requires about 2600 feet of gravel-packed 12-inch drill hole, 40 five-foot well screens, and 40 pumps of 0–100 gpm capacity. In order to simulate the effects of the fully constructed ditch it will be necessary to pump the wells continuously year round. As with previous section 1) this scheme may be implemented in a series of increments of such length as to have the end of the row of wells at least 4 years ahead of overburden removal. These shorter increments would allow reuse of some of the equipment. As with 1) above, this increment approach is less efficient in terms of total water removal and will have implications for later facets of the program.

The second division is designed to lower water levels in the central portion of the lease, particularly those areas where depressions in the tar sands surface do not allow rapid drainage to the ditch or row of wells. This division can be implemented in either of two ways:

1) Through the use of about 37 dewatering wells requiring a total of 2270 feet of drill hole. These wells would be installed two years before excavation was to reach a specific locale if piezometric data indicate

that significant lowering of the water table has not taken place. It is expected that there might be about 6 of these wells operating at any one time.

It is likely that if the first division recommendations are instituted in full (rather than by the increment approach) many of these wells would not be necessary. It is also likely that they would have to be pumped only during spring, summer, and fall if the full ditch or row of wells was in existence. The wells should be pumped year round if the incremental approach of division use is selected.

2) Through the construction of a second ditch and modifications to the main ditch. This plan calls for a ditch 11,600 feet long trending northwest-southeast through the center of the lease and modifications to the main ditch making it 16,800 feet long. Both ditches would average 60 feet deep. Water may be discharged from the ditches by one of two methods:

a) through the use of pumps with expected capacity of 1 to 5 mgd, or

b) through the construction of two outlets, 1800 and 4000 feet long, to carry water from the east and west ditches, respectively, to the escarpment along the Athabasca River.

It is expected that this second alternative to the second division represents the best possible dewatering procedure but there is also the feeling that it may be more than is needed.

My recommendations are:

1. Install piezometers where indicated and monitor water levels monthly.
2. Drain the muskeg via closely spaced ditches.

3. Install and pump withdrawal wells 2 years before excavation in the Stage I (pre-1976 operations) area where need is indicated by piezometers.

4. Construct main ditch and outlet immediately.

5. Install Stage II (post-1976 operations) auxilliary wells as needed 2 years before excavation.

HYDROGEOLOGY

GEOLOGY

The geology of the lease consists basically of glacial drift overlying Cretaceous and older consolidated materials. Bitumen accumulations have taken place in the McMurray and Clearwater Formations and to some extent in the glacial till. Economical bitumen accumulations occur dominantly in the McMurray. The Devonian limestones, lying immediately below the McMurray Formation, appear to be quite dense and massive with little if any oil accumulation.

The glacial overburden has been divided into members and submembers by Linckens (1965). For my purposes, I feel that it is better to classify the materials into hydrogeologic units through a combination of grain size and assumed hydraulic conductivity. Twenty-three cross sections of the lease were drawn, all trend northeast-southwest and are located along major core hole lines. The purpose of these sections was to reveal the extent of individual hydrogeologic units and thus to aid in the placement of wells and ditches.

Based on these cross sections, it appears that sands and gravels (high hydraulic conductivity) are most abundant in the southern and far northern portions of the lease while finer-grained (low hydraulic conductivity) materials dominate the central portion.

The effects of the high hydraulic conductivity materials in the southern portion of the lease have already been observed through the widespread decline of the water table due to the existing sump. This widespread effect cannot be expected to continue as excavation moves northwestward and in fact it appears that the water table rises quite rapidly northward from the ditch.

There is some doubt in my mind as to the nature and extent of "sand" deposits noted in the core hole logs furnished me. These sample descriptions were made for geologic rather than hydrogeologic purposes and hence inferences on hydraulic properties are tenuous. I have assumed that "sand," as noted in the logs, represented a significantly higher hydraulic conductivity than that noted as "clay."

It is my understanding that all materials containing less than 8% bitumen are classed as overburden. These are also the materials which must be dewatered in order to be removed. It is apparent from observation and from previous work (Clark, 1959) that the hydraulic conductivity of a soil specimen diminishes quite rapidly as the amount of saturation with bitumen increases. My approach was to concentrate on dewatering the overburden containing little or no bitumen and to incorporate, where possible, incidentals which would aid in dewatering the uneconomical tar sands.

AQUIFER TESTING

Appendix A presents the data from various aquifer performance tests conducted in the lease between 1967 and 1970. I regard the content of information in my possession regarding these tests as less than satisfactory.

Table 1 summarizes the results of the aquifer testing. It can be seen that the values of hydraulic conductivity vary over quite a wide range. It appears that these values tend to range between about 200 gpd/sq ft and 5 gpd/sq ft. According to Todd (1959) this is a reasonable range of values for sands, silts, and tills and would be classified between poor and good aquifers. There may be scattered aquifers with hydraulic conductivities greater than 200 gpd/sq ft but they do not appear to be abundant. The occasional occurrence of better aquifers will aid the dewatering

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Tp 92, R10, W of 4th M.

Table 1. Summary of Aquifer Testing

Lsd.	Sec.	Piezometer No.	T(gpd/ft) P	T(gpd/ft) R	M (ft)	D (ft)	K(gpd/sq ft) P	K(gpd/sq ft) R	Comments
		Bail Tests:							
1	15	P 19 74D/13E	-	35,000	48	15		720	Data scattered
		P 19	34,000	32,000	48	15	710	660	Data scattered, 85 minutes bailing
15	22	P 29 74E/4E	-	53	10	5		5	Late data plot good line
15	27	P 39 74E/4E	20,000	-	19	6	1,000	-	Very poor; data scattered, 55 minutes bailing
7	22	P 48 74D/13E	180	820	24	7;18	7	34	Fair, straight line fit to data
2	22	P 49 74D/13E	-	13	24	26	-	0.5	Very poor, data scattered
		P 49* (P50)	2,500	6,100	24	26	110	260	Fair, straight line fit to data
15	22	P 51 74D/13E	1,300	2,000	29	0;24	46	70	Fair, straight line fit to data
9	28	P 54 7E/4E	750	1,200	15	0	50	81	Fair, straight line fit to data
15	22	P 55* 74D (P51)	1,300	1,000	22	0;25	57	4,800	Recovery data questionable
9	28	P 56 74E/4E	-	9,000	27	4;23	-	330	Good straight line fit to data
12	27	P 57 74E/4E	2,500	2,700	21	6	120	130	Good straight line fit to data
1	28	P 58 74E/4E	980	2,800	50	0	20	56	Pumping data poor; recovery fair
		Pumping Test:							Pumping too variable after 2800 minutes
13	14	CH 117 74D/13E	21,000	8,000	45	6	460	180	Good straight line fit to data
12	14	Obs.#1 "	7,600	3,900	45	6	170	86	Good straight line fit to data
29	"	Obs.#2 "	13,000	11,000	45	6	280	240	Possible break in recovery curve
				3,700				82	

EXPLANATION

P - pumping phase
R - recovery phase
T - transmissibility
M - total saturated thickness of apparent aquifers
D - depth to tops of aquifers
K - hydraulic conductivity K = T/M
Results presented to two significant figures

*While pumping well number in parentheses.

but they do not appear to be abundant enough to raise the expected upper limit of hydraulic conductivity.

PRECIPITATION AND EVAPOTRANSPIRATION

Table 2 depicts adjusted potential evapotranspiration and precipitation at Fort McMurray airport. Mean annual precipitation at the airport is about 17 inches while adjusted potential evapotranspiration averages about 19 inches per year. This would seem to some to indicate that all precipitation on the lease is removed through evapotranspiration. This is not necessarily the case since the distribution of precipitation is not uniform throughout the period when evapotranspiration is possible. A great excess of water occurs in the spring due to snowmelt and high rainfall while only limited evapotranspiration takes place; this provides abundant amounts of water for infiltration. It is shown in Table 2 that not until about July of the average year does cumulative evapotranspiration equal the amount of water available from snowmelt and spring precipitation (cumulative precipitation from November). Thus, from spring thaw until early summer water will infiltrate the overburden. Much, but not necessarily all, of this will subsequently be transpired during later portions of the summer. The amount not transpired contributes to the local groundwater flow and sustains the water table.

Table 2. Temperature, Precipitation, and Adjusted Potential Evapotranspiration at Fort McMurray Airport

MEAN ANNUAL DATA

	Month												Average or Total
	J	F	M	A	M	J	J	A	S	O	N	D	
Temp. °F	-6.3	1.0	15.3	34.8	48.9	55.9	61.6	58.3	48.3	36.7	16.5	0.5	31.0
Adj. Pot. Et.(1)	0	0	0	0.69	2.79	3.67	4.93	4.12	2.23	0.55	0	0	18.98
Cumulative Adj. Pot. Et. for Year	0	0	0	0.69	3.48	7.15	12.08	16.20	18.43	18.98	18.98	18.98	
Precipitation(2)	0.84	0.65	0.88	0.75	1.31	2.36	2.93	2.36	1.93	1.03	0.93	0.88	16.85
Cumulative Precipitation from November	- / 2.65	- / 3.30	- / 4.18	- / 4.93	- / 6.24	- / 8.60	- / 11.53	- / 13.89	- / 15.82	- / 16.85	0.93 / -	1.81 / -	

(1)Calculated by Thornthwaite (1959) Method at heat index of 25. These figures are slightly high since the true heat index is 21.1 but the lowest heat index in the Thornthwaite tables is 25.

(2)Canada Department of Transport, Meteorological Branch (1967).

DEWATERING PHILOSOPHY

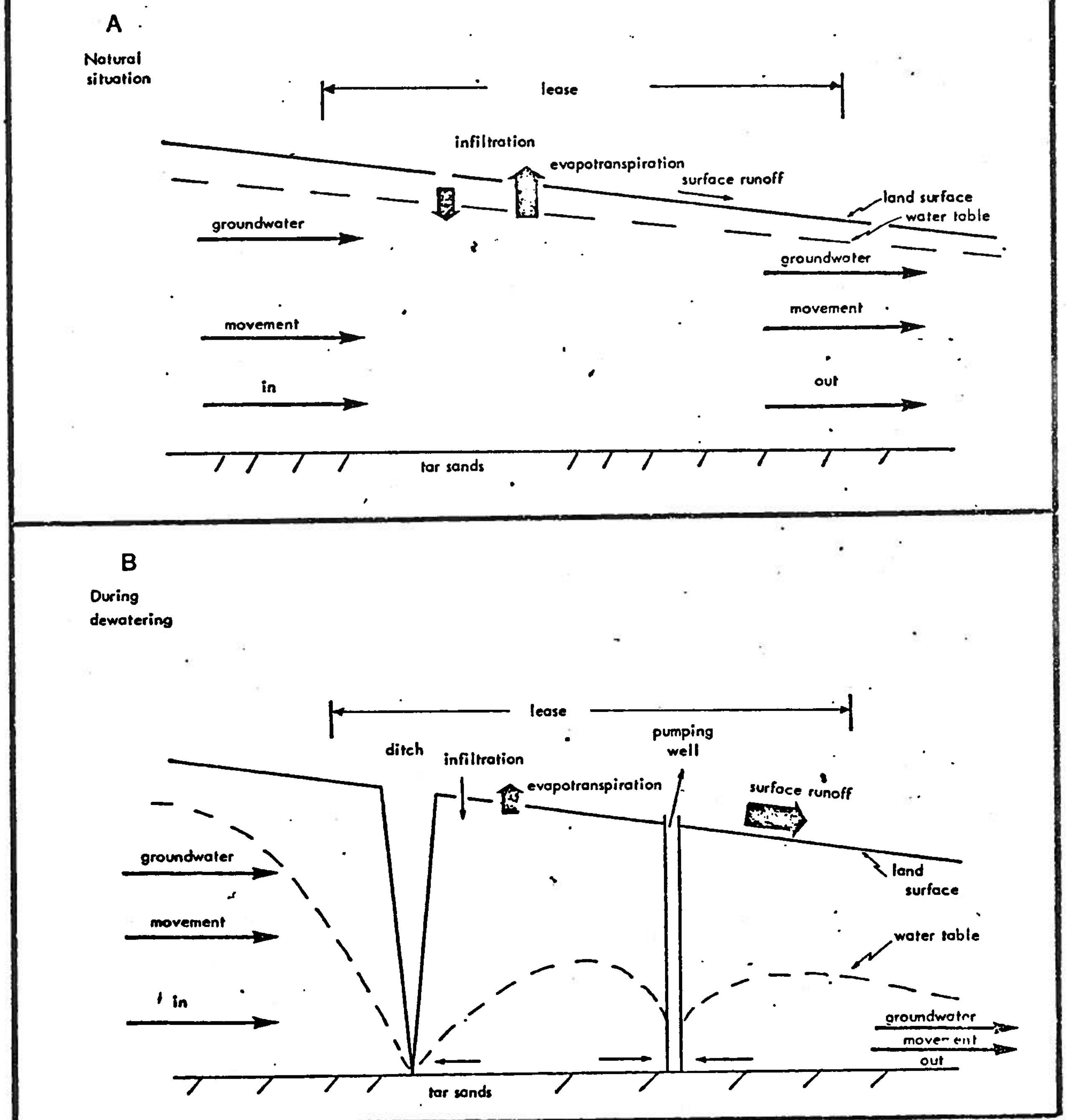
Figure 1-A shows an idealized and simplified cross section of the G.C.O.S. lease depicting the basic components of the hydrologic cycle. Very simply, the height of the water table is a function of four parameters: 1) groundwater movement into the lease, 2) groundwater movement out of the lease, 3) infiltration from precipitation, and 4) evapotranspiration. Changes in the elevation of the water table will result from changes in any of these four components. The dewatering scheme outlined in this report acts to articially reduce 1) and 3) and to increase 2), thereby lowering the water table. There will be a coincidental decrease in evapotranspiration as a result of these changes but its influence will be negligible.

Figure 1-B shows the theoretical result of the above-mentioned changes. Inflowing groundwater is intercepted and removed by discharging water from a ditch or well at the "up gradient" edge of the lease. There is also flow toward this discharge point from the interior of the lease. Groundwater flow out of the lease continues but diminishes with time as the water table declines. The pumping well also effects a lowering of the water table by removal of stored water. Infiltration from precipitation is reduced by increasing runoff as outlined at a later point.

The net effect of the above is a general decline in the elevation of the water table. The rate of this decline is a function of the effectiveness of each artificial change in the hydrologic regime; it is also dependent on the hydraulic conductivity of the overburden material and of time. The greater the hydraulic conductivity the more rapid will be the water table decline — all other factors being equal. Since pumping and groundwater outflow are volume/time relationships, it is

FIGURE 1

Schematic Cross Sections of the Hydrologic Cycle at the G.C.O.S. Lease



obvious that the water level decline will be directly related to time.

Rates of pumping (from wells or ditches) are limited by hydraulic conductivity and available drawdown. Since these two factors are fixed at all points in the lease, time becomes a major consideration. To illustrate, physical characteristics of the aquifers predetermine the rate at which a well (or ditch) can be pumped (and still yield water) over extended periods of time; thus the variable left for manipulation becomes time. The longer and more continuously a well or ditch is pumped the farther away will it exert an influence to lower the water table.

QUANTITATIVE ANALYSIS OF DEWATERING

General Theory

Two Ditches

According to Glover (1964) the height of the water table between two ditches which have been instantaneously dewatered can be approximated by:

$$h \doteq \frac{4D}{\pi} \sum_{n=1,2,3}^{n=\infty} \left[\frac{e(\exp \frac{-n^2 \pi^2 \alpha t}{L^2}}{n} \right] \sin \frac{n \pi x}{L}$$

where:
- h = height of water table above bottom of ditch at time (t);
- D = drainable depth of aquifer (equal to ha t start of dewatering);
- t = time;
- L = distance between ditches;
- x = horizontal distance to h;
- $\alpha = \frac{3KD}{2V}$ for fully dewatering ditches;
- V = specific yield;
- K = hydraulic conductivity.

This approximation has limitations and probably is not adequate at short distances (100 feet?) from the ditches but will be reasonably valid at larger distances when the concern for dewatering is greater.

Single Ditch

According to Glover (1964) the head at any distance from a single ditch that has been instantaneously dewatered may be approximated by:

$$h = \frac{2D}{\sqrt{\pi}} \int_{0}^{\frac{x}{\sqrt{4\alpha t}}} e^{-u^2} du.$$

The flow into the ditch is approximated by:

$$F = \frac{KD^2}{\sqrt{\pi \alpha t}}$$

where all terms are as defined previously. These expressions are also less accurate at relatively short distances from the ditch and become more accurate with distance.

Wells

Glover (1964) states that the drawdown due to a single well can be approximated by:

$$s = \frac{Q}{2\pi DK\sigma} \left(1 - \sqrt{1 - 2\sigma \int_{\frac{r}{4\alpha t}}^{\infty} \frac{e^{-u^2}}{u} du} \right)$$

where: s = drawdown at any point at any distance r from the pumping well;

Q = rate of pumping;

$\sigma = \dfrac{Q}{2\pi KD^2}$;

and other terms as defined previously.

This expression is also limited in that it is not as good an approximation to the actual drawdown close to the well as it is at large distances. This is no great drawback, however, since we are concerned about effects at large distances anyway.

Application to Lease

Water Levels

It should be borne in mind that all water levels and discharges calculated in this section are advisory only. They are presented to show only the probable range of values to be expected under conditions existing in the lease.

The calculations in this section utilize the following constants:

Hydraulic conductivity: K = 200 gpd/sq ft (maximum) $\cong$ 32 ft/day

= 6 gpd/sq (minimum) $\cong$ 1 ft/day

Specific yield: V = 0.15

Drainable depth of aquifer: D = 50 feet.

Single Ditch

Table 3 shows the water levels which might be expected at various distances and times caused by a sudden lowering of the water level in the main ditch (see Map 1). I felt that these water levels could best be calculated by using an imaginary ditch 5000 feet northeast and parallel to the main ditch. Thus calculations actually are made with the equations for a two-ditch system. These ranges of water levels might be expected to occur to the northeast of the main ditch.

Figure 2 shows plots of flow into the ditch from the northeast and infiltration. The infiltration curves are not the same for both maximum and minimum K-values since the amount of infiltration is a function of the area of influence of the ditch at any one time. For an infiltration rate of 0.3 cubic feet/year/sq foot the equilibrium time (the time from the start of dewatering until recharge from infiltration is sufficient to sustain the water level configuration then existing) for a K-value of 200 gpd/sq ft is about 260 days. From Table 3 it can be seen that at

Table 3. Water Levels Due to Main Ditch and Discharge into Main Ditch

Interior Flow from Northeast Only — Simulated by Imaginary Ditch
5000 feet Northwest (Distance to Escarpment)

$K = 200\ gpd/ft^2$

Time (days)	h at various x northeast of main ditch					Discharge (cubic ft/day/ft)
	500	1000	1500	2000	2500	
10	32	46	50	50	50	32
50	16	28	38	44	46	27.1
100	12	20	27	32	34	14.8
150	8	15	20	28	25	8.0
200	6	10	14	17	18	4.1
300	3	6	8	9	10	1.3
500	1	2	2	3	3	.12

$K = 6\ gpd/ft^2$

Time (days)	500	1000	1500	2000	2500	Discharge (cubic ft/day/ft)
10	50	50	50	50	50	1.0
50	48	50	50	50	50	1.0
100	44	50	50	50	50	1.0
150	40	50	50	50	50	1.0
200	36	48	50	50	50	1.0
300	32	46	50	50	50	1.0
500	26	42	48	50	50	1.0
1000	19	34	43	48	48	.92
2000	14	26	35	40	42	.71
4000	9	17	22	28	29	.34
6000	6	12	16	18	20	.16
8000	4	8	11	12	13	.07

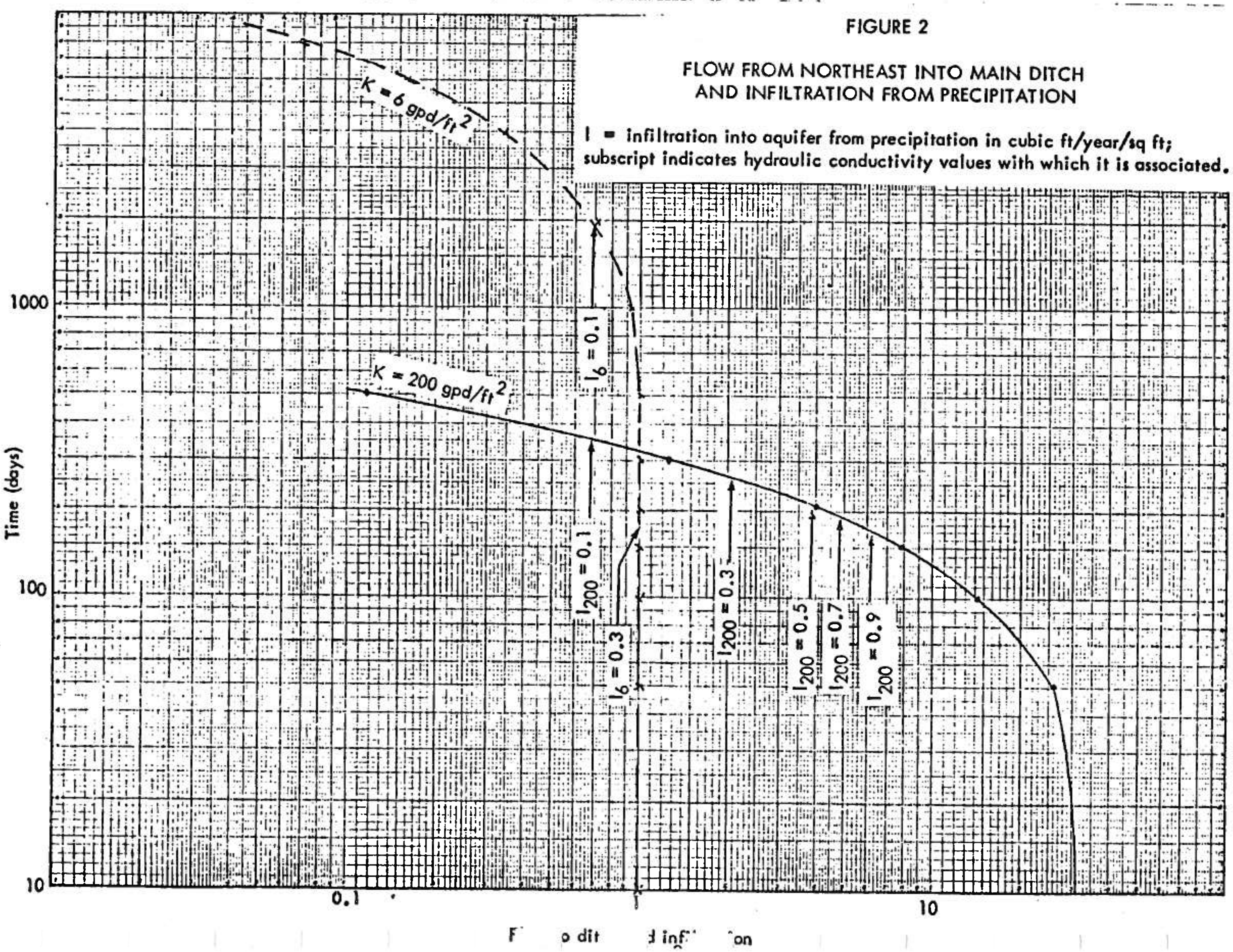


FIGURE 2

FLOW FROM NORTHEAST INTO MAIN DITCH
AND INFILTRATION FROM PRECIPITATION

I = infiltration into aquifer from precipitation in cubic ft/year/sq ft; subscript indicates hydraulic conductivity values with which it is associated.

this time the aquifer will be effectively dewatered for a distance of over 2500 feet northeast of the ditch. At a K-value of 6 gpd/sq ft the equilibrium time for the same rate of infiltration is 200 days. Table 3 reveals that the aquifer is really only very slightly dewatered in this time and thus the dewatering is fairly ineffective. If the reader performs this same analysis for infiltration rate of 0.1 cubic feet/year/sq ft, it will be demonstrated that as infiltration is reduced to very low values the effect of a dewatering scheme is enhanced.

Table 4 shows groundwater levels due to an instantaneous dewatering of a single isolated ditch and the resultant flow caused by that dewatering. These levels can be considered as the range of levels that might occur to the southwest of the main ditch. It can be seen that at the high value of hydraulic conductivity the area of large drawdowns becomes quite extensive in a relatively short period of time. In contrast, the low hydraulic conductivity values do not allow for very efficient dewatering at any great distance, even in 4000 days.

Figure 3 depicts the time variance in flow to the main ditch from the southwest and the time variance of infiltration into the area of influence of the ditch. For an infiltration rate of 0.3 cubic feet/year/sq foot, equilibrium times of 1100 and 260 days are obtained for K-values of 6 and 200 gpd/sq ft, respectively. The equilibrium time for the higher hydraulic conductivity is satisfactory as seen in Table 4 but that for the lower hydraulic conductivity represents significant dewatering only to a distance of about 600 feet.

If infiltration is reduced to 0.1 cubic feet/year/sq ft, then the equilibrium times on Figure 3 increase to 2000 and 350 days. For low K-values the 2000 days represents adequate dewatering out to about 1000 feet from the ditch. The 350

Table 4. Water Levels Due to a Single Ditch

Time (days)	h (feet) at various x southwest of ditch 500	1000	1500	2000	Minimum x for h 50 (feet)	Flow to ditch cubic ft/day/ft of ditch
K = 200 gpd/ft^2; all other parameters outlined in text						
10	28	44	49	50	2,000	137
100	10	18	27	33	5,000	44
300	6	11	16	21	8,000	25
600	4	8	12	15	12,000	18
1000	3	6	9	12	15,000	14
K = 6 gpd/ft^2; all other parameters outlined in text						
10						4.86
100	42	50	50	50	1,000	1.54
300	28	44	49	50	2,000-	0.89
600	21	38	46	49	2,000+	0.63
1000	17	30	40	46	2,500	0.49
2000	12	23	32	39	4,000	0.35
3000	10	19	27	34	5,000	0.28
4000	9	17	24	31	6,000	0.24

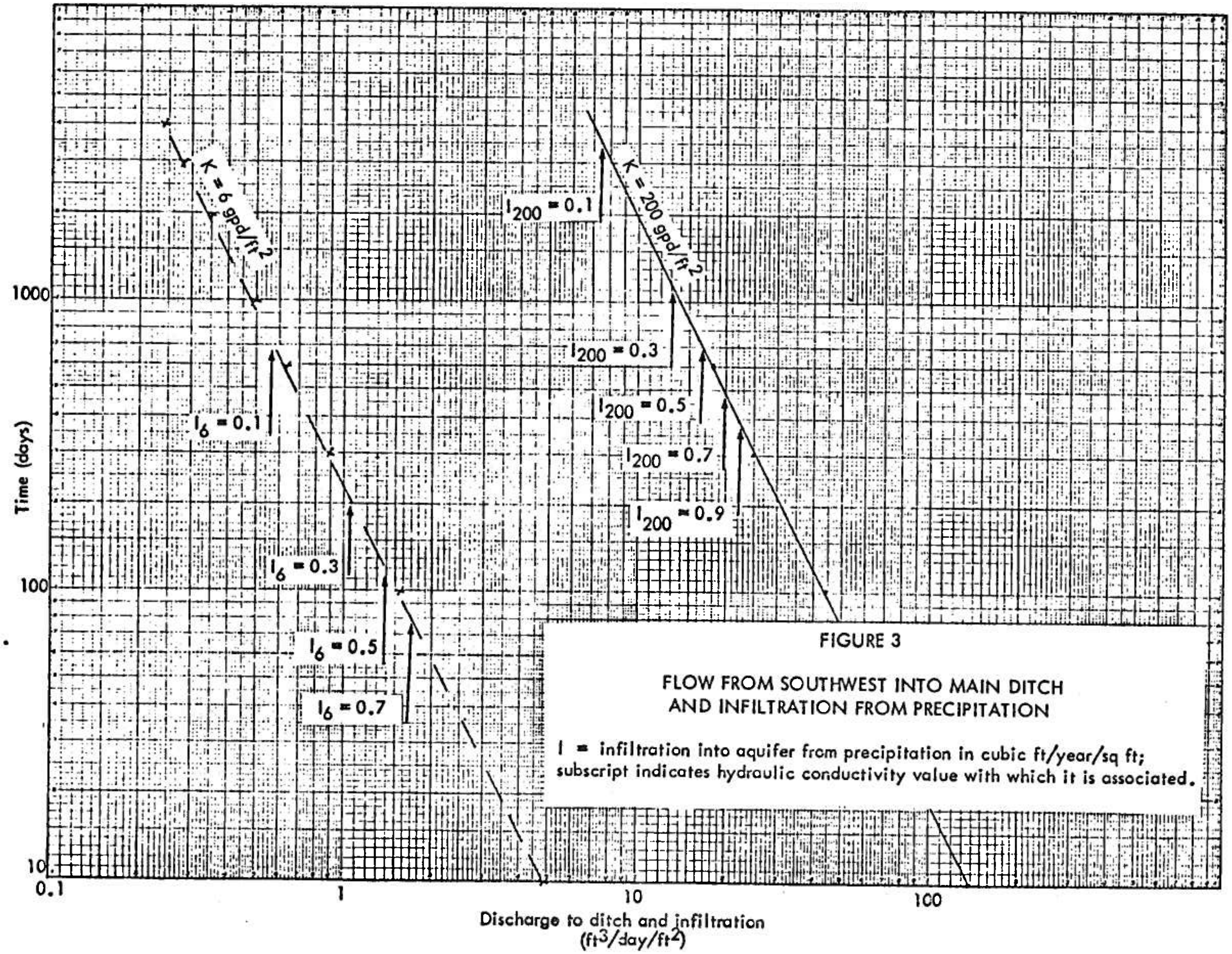


FIGURE 3

FLOW FROM SOUTHWEST INTO MAIN DITCH AND INFILTRATION FROM PRECIPITATION

I = infiltration into aquifer from precipitation in cubic ft/year/sq ft; subscript indicates hydraulic conductivity value with which it is associated.

days for K-values of 200 gpd/sq ft represent adequate dewatering out to about 2000 feet. The important point is that by controlling the amount of infiltration the effect of a dewatering scheme becomes greater.

Two Ditches

Table 5 was produced using the Glover equation, the given parameters and a distance of 3000 feet between ditches (see Map 2). The design was intended to be as valid as possible but the values obtained should be viewed as order-of-magnitude only. The values are symmetrical about the midline between ditches so only values of x to 1500 feet distance are presented.

The water levels given in this table represent probable extremes ranging from a possible high hydraulic conductivity to a probable lower limit, since most materials encountered on the lease should have hydraulic conductivities in the range between these two extremes. It should be kept in mind, however, that the water levels depicted on Table 5 do not consider infiltration into the system. Figure 4 shows a plot of flow to the ditch (from table 5) versus time; it also shows various constant amounts of infiltration into the 1500 foot by a foot segment of aquifer which is contributing to flow into the ditch. When these two values are equal an equilibrium situation exists, with discharge equal to recharge and cessation of decline in water levels. Thus, if infiltration amounts to 0.3 cubic foot/year/foot of aquifer the aquifer with a K-value of 200 gpd/sq ft will reach equilibrium water levels in 125 days. As seen on Table 5 these levels are satisfactory since most of the original saturated thickness has been dewatered. On the other hand, the aquifer with a K-value of 6 gpd/sq ft will have equilibrium water levels of those existing on the 700th day. As seen from Table 5 these water levels do not represent an acceptable

Table 5. Water Levels Between Two Ditches

K = 200 gpd/ft^2; other parameters outlined in text

Time (days)	Values of h (feet) at various x: 300	600	900	1200	1500	Flow to ditch cubic ft/day/ft of ditch
10	20	35	45	47	49	51.4
50	8	15	21	25	26	15
100	4	6	9	10	11	2.58
150	2	3	4	6	6	0.77
200	1	2	2	2	2	0.085

K = 1 gpd/ft^2; other parameters outlined in text

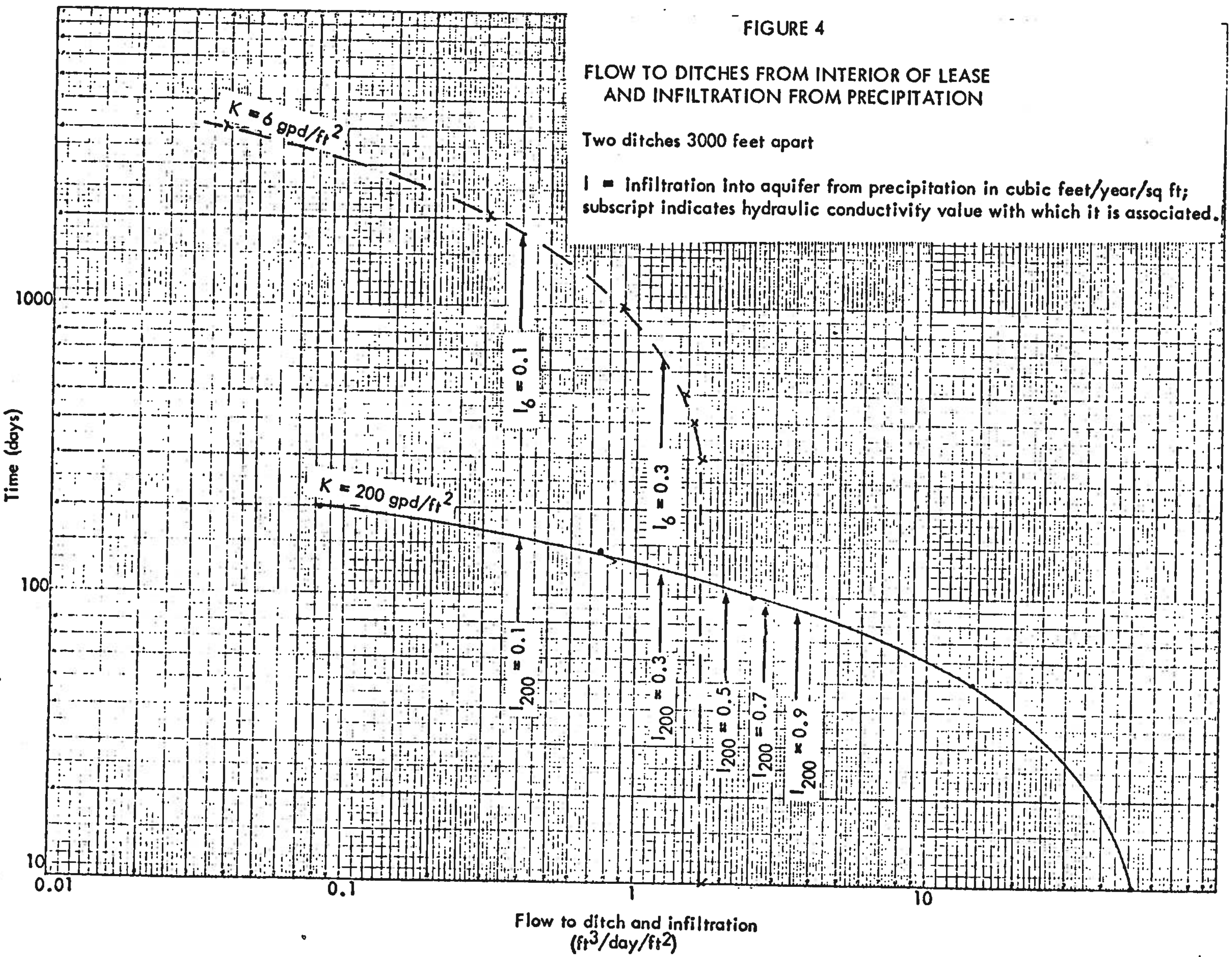
Time (days)	300	600	900	1200	1500	Flow to ditch cubic ft/day/ft of ditch
10	50	50	50	50	50	1.67
50	41	49	50	50	50	1.67
100	33	48	50	50	50	1.67
200	25	41	48	50	50	1.67
300	20	35	44	48	49	1.64
400	18	33	42	47	48	1.54
500	16	30	39	45	46	1.45
600	15	28	37	42	44	1.33
1000	12	22	30	35	36	0.87
2000	7	12	18	20	21	0.31
4000	2	4	6	7	8	0.038

FIGURE 4

FLOW TO DITCHES FROM INTERIOR OF LEASE AND INFILTRATION FROM PRECIPITATION

Two ditches 3000 feet apart

I = infiltration into aquifer from precipitation in cubic feet/year/sq ft; subscript indicates hydraulic conductivity value with which it is associated.



amount of dewatering. If infiltration can be reduced by some means then the equilibrium time will become greater and will allow a greater dewatering of the aquifer.

Water levels to the southwest of the western ditch are represented by Table 4, Figure 3, and the discussion pertaining to them.

Wells

The Glover equation for water levels at any distance from a pumping well in an unconfined aquifer was utilized to construct Table 6.

This table is perhaps not as useful a guide as were Tables 3, 4, and 5 since values in it will change with changes in pumping rates. It does serve to show, however, that a good deal of influence can be exerted out to distances of 200 to 300 feet in time periods ranging from 600 to 2000 days, depending on the hydraulic conductivity.

The effect of a row of wells can be derived from Table 6 (with some qualifications). The water level drawdown at any point in an aquifer is the sum of the drawdowns at that point caused by each individual well. Thus, for a number of wells in a straight line, the drawdown midway between each may be arrived at by doubling the drawdown caused by one well at a distance equal to half the well spacing. For instance, for K = 200 gpd/sq ft and a spacing between wells of 400 feet, the total drawdown halfway between the wells in 100 days would be 16 feet (twice the 8-foot drawdown due to one well at that time).

If a good knowledge of the distribution of hydraulic conductivity within the lease existed then it would be possible to calculate the necessary spacing between wells given the required time constraints. Since this knowledge does not exist, it is

Table 6. Water Levels Due to a Pumping Well

$K = 200\ gpd/ft^2$, $Q = 150\ gpm$; other parameters as defined previously						
Time (days)	h (feet) at various r (feet)					
	5	100	200	300	500	700
10	29	34	37	39	41	42
100	21	39	42	44	46	47
300	15	35	40	41	43	45
600	13	35	38	40	42	43
1000	8	16	23	26	29	32
$K = 1\ gpd/ft^2$, $Q = 6\ gpm$; other parameters as defined previously						
10	35	49	50	50	50	50
100	26	45	48	50	50	50
300	21	42	46	47	49	50
600	16	41	44	46	48	49
1000	13	39	43	45	47	47
2000	5	37	41	43	46	47
4000	0	35	39	41	44	46

necessary to choose the well spacing most likely to produce the required drawdowns in a reasonable time and considering a practical distance for well spacing.

Discharge Rates

Flow into Single Ditch

Using Figure 2 for flow from the northeast, Figure 3 for flow from the southwest, and a main ditch length of 14,500 feet, it is possible to calculate the approximate amount of pumping required to keep the ditch dewatered for various infiltration values. Curves representing these values are presented in Figure 5. It must be kept in mind that the analysis used to arrive at this point assumed that the ditch was completely full of water and then instantaneously emptied. The values in Figure 5 are those necessary to keep the ditch empty once the above phenomenon has occurred. It is emphasized that the early discharge values in the tens of millions of gallons per day are not realistic to the actual operation of the G.C.O.S. main ditch. The more practical approach is to initiate pumping at about 5 mgd, pump the ditch dry, and then continue pumping at a rate just low enough to maintain water in the actual pumping sump. The long-term pumping rate might conceivably be between 1.0 and 2.0 mgd. Figure 5 is intended to give ranges of pumping values based on the maximum and minimum hydraulic conductivity values and infiltration rates expected to occur in the lease. In addition, since ditch excavation is a slow process, it will be possible to start pumping after only a few hundred feet of construction has been accomplished. It should require only low (1 mgd) pumping rates to keep the ditch dry as construction proceeds.

The effects of increasing infiltration are evident in Figure 5. As infiltration increases the equilibrium time declines and discharge rates necessary

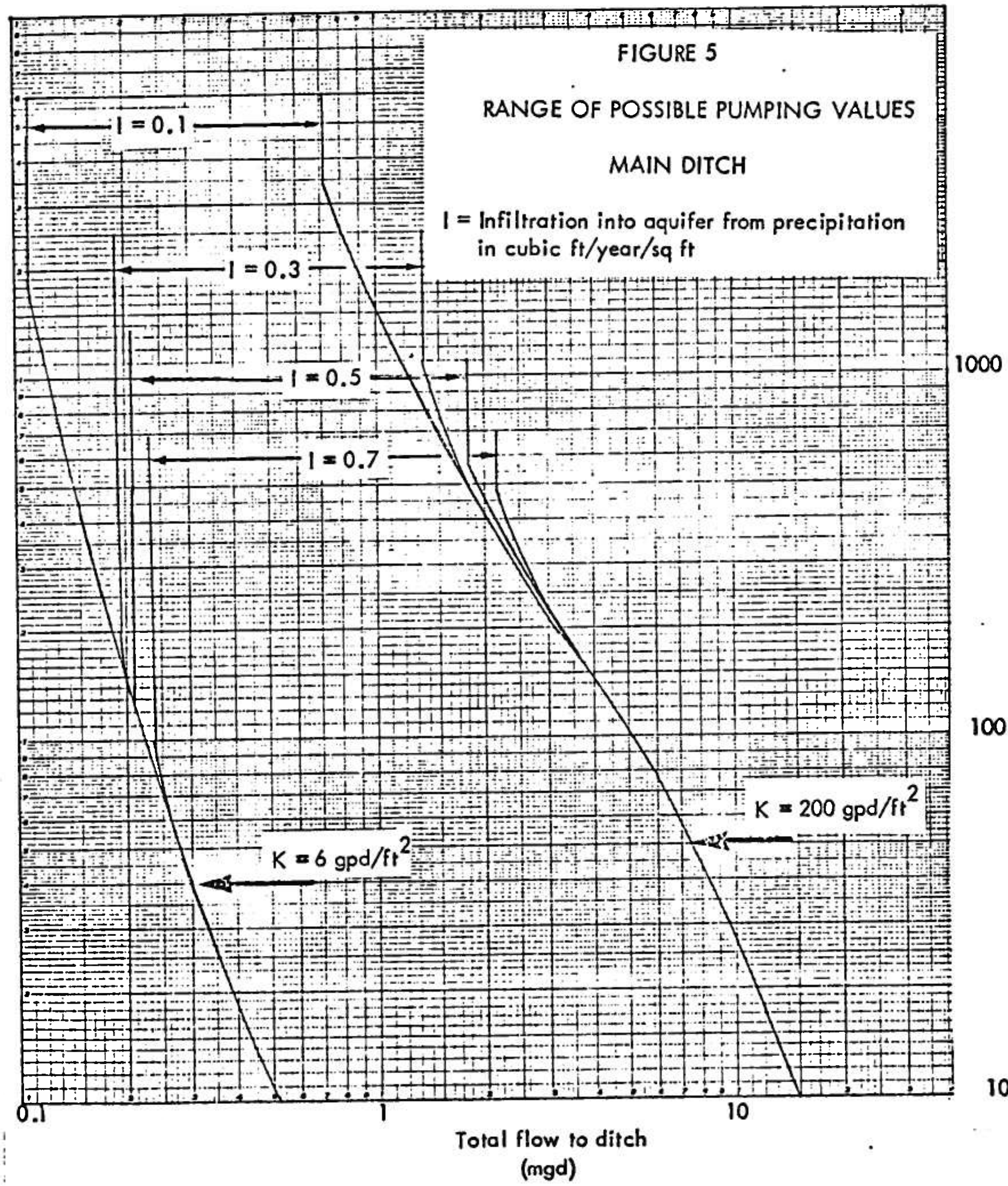


FIGURE 5
RANGE OF POSSIBLE PUMPING VALUES
MAIN DITCH
I = Infiltration into aquifer from precipitation in cubic ft/year/sq ft
I = 0.1
I = 0.3
I = 0.5
I = 0.7
K = 200 gpd/ft²
K = 6 gpd/ft²
1000
100
10
Time (days)
0.1
1
10
Total flow to ditch (mgd)

to keep the ditch dry increase. For instance, for a K-value over the entire lease of 200 gpd/sq ft the equilibrium pumping rate and time are 0.85 mgd and 3300 days, respectively, for infiltration of 0.1 cubic ft/year/sq ft. The same values for infiltration of 0.7 cubic ft/year/sq ft are 2.6 mgd and 470 days. It is very likely that the increased pumping costs and decreased efficiency of dewatering will be more than compensated for by efforts to reduce infiltration.

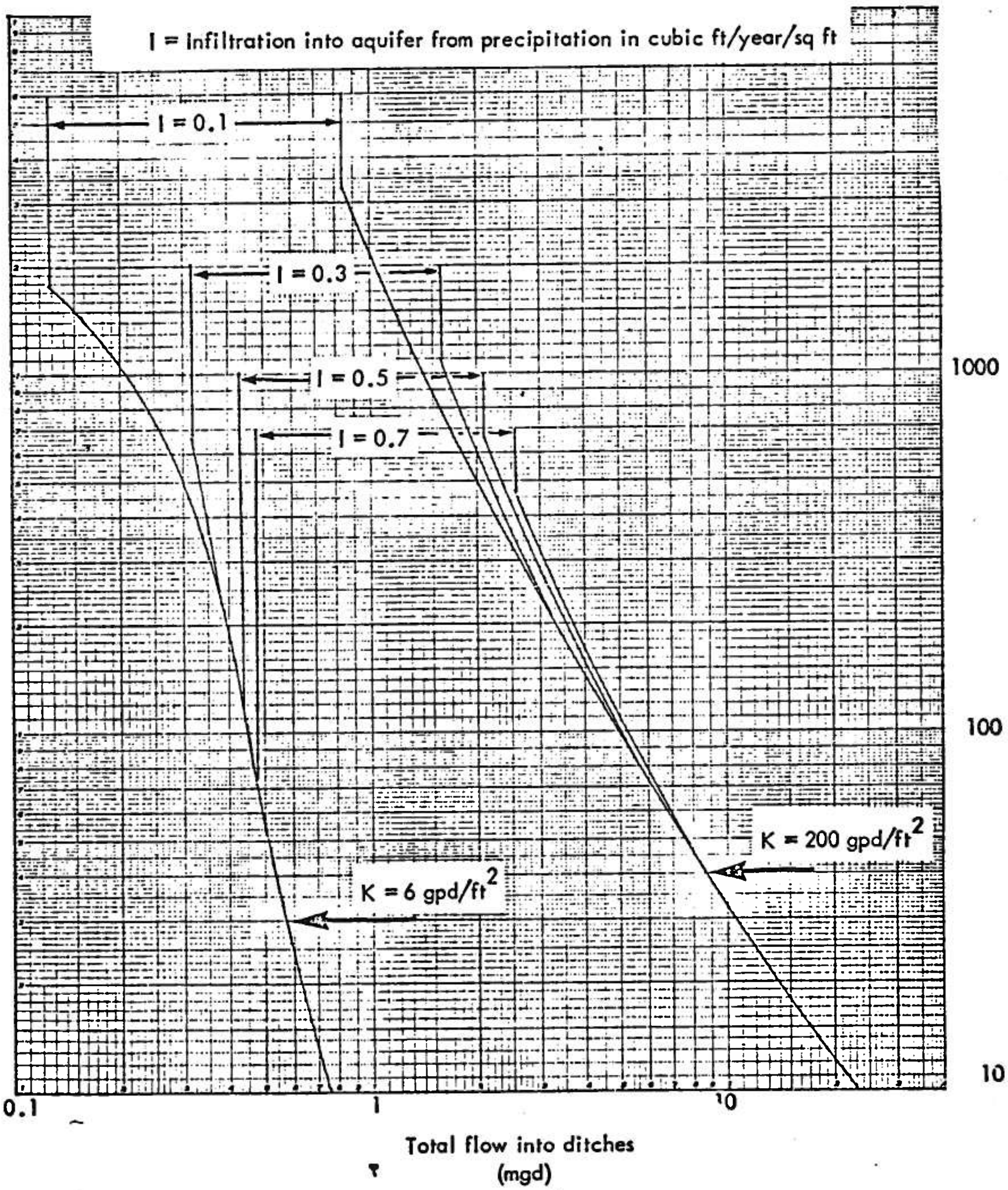
Flow into Two Ditches

Using Figure 3 for flow into the western ditch from the southwest, Figure 4 for flow from the interior of the lease into both ditches, a west ditch length of 16,800 feet, and an east ditch length of 11,600 feet, it is possible to calculate the rate of pumping required to keep the two ditches dewatered for various infiltration values. It was assumed that flow into the eastern ditch would rapidly decline to insignificant amounts and so was not included in the pumping value calculations. Figure 6 presents these values. Here again, the limitations discussed in the last section are valid. The figures are supposed to represent ranges of pumping values which may be necessary given the expected hydraulic conductivities of sediments in the lease. It is expected that two pumping stations with initial capacities of 5 mgd each would be necessary to keep both ditches empty. With time, the necessary pumpage from each ditch might drop to somewhere between 0.5 and 1.0 mgd.

As in the previous set of figures it is seen that the amount of infiltration decreases the equilibrium time and increases the necessary pumpage.

FIGURE 6

RANGE OF POSSIBLE PUMPING VALUES - TWO DITCHES



PIEZOMETER NETWORK

Piezometers provide the means for measuring the elevation of the water table and as such are a necessary part of any dewatering scheme. The effectiveness of any dewatering scheme is best evaluated through the judicious monitoring of piezometers. There is no other means as inexpensive as piezometers whereby the elevation of the water table can be measured. Without this information one can only guess whether the dewatering scheme is effective or not. The dewatering schemes I have prepared are based on water level measurements in piezometers; the program cannot be expected to perform properly and economically without this information. The entire piezometer scheme outlined herein is necessary no matter which of the laternative dewatering schemes are selected. The entire network should be brought up to specified conditions as soon as possible.

Existing Piezometers

Map 3 presents information on existing and proposed piezometers. Based on the information presented it can be seen that many existing piezometers are suitable for measuring large declines in the water table elevation; that is, they very nearly fully penetrate the overburden. A listing of existing piezometers which are judged as terminating at too shallow a depth is presented on Map 3. The basis for the selection of these specific wells consisted of a combination of location, expected drawdown, and the existence of nearby piezometers. These piezometers should be brought up to specifications as soon as possible.

Proposed New Piezometers

Five new piezometers are deemed necessary; their location is shown on Map 3. These sites were selected due to the lack of data in certain areas of the lease.

Construction of Piezometers

Both the new and the renovated old piezometers can be constructed of 1 1/4 inch black pipe with a short screened sandpoint at the end. There will be a need for about 1200 feet of pipe and 15 sandpoints. The top of the well should be high enough to be above the usual snow level to facilitate winter measurement.

Frequency of Monitoring

The importance of water level measurements has been stressed previously. Measurements should be taken monthly in all piezometers. The water in those piezometers in which the level is more than 4 or 5 feet below the surface will probably not freeze during winter and monthly measurements should continue all year round. Piezometers in which the depth to water is less than 5 feet can be "frost-proofed" by pouring a half gallon of heavy motor oil into each one; this should depress the water level by about nine feet. The level of the surface of the oil will be approximately the elevation to which the water would rise were it not for the oil on top of it.

DRAINAGE OF MUSKEG

Purpose

In order to reduce infiltration of precipitation into the lease it is desirable to drain the muskeg. Rapid removal of water from the surface of the land leaves less water to infiltrate and will thus cause a lowering of the water table. The dependence of groundwater levels on infiltration has been demonstrated.

The major benefits of proper drainage of the muskeg will accrue over long periods of time, the possible result being the need for fewer auxilliary wells than are now predicted. If the whole lease is immediately covered by an adequate muskeg drainage system it may well be that its combination with the main ditch would result in a need for less of the auxilliary wells now predicted for the post-1980 excavations.

I regard this muskeg ditching as a necessary part of the dewatering procedure which should be implemented in total as soon as possible.

Method

Exact planning and layout of muskeg drainage ditches was not carried out as part of this report. It appears from the topographic map that shallow, gravity flow (2-3 feet) ditches could be constructed utilizing many of the existing cutlines on the lease. Most effecient drainage would be accomplished by ditches with carefully planned layouts; this would involve extensive new cutlines. Considering the low permeability and great water retention capabilities of muskeg it would be advisable to construct these ditches at a spacing of no more than 900 feet.

Actual construction of the ditches can be undertaken with a small dragline, however the magnitude of the total system may warrant specialized equipment. I am not familiar with any specific muskeg ditching equipment and there is probably none available in Western Canada. The Finnish, Swedish, and Germans have developed specialized muskeg equipment and it may be available in Eastern Canada.

It is expected that the ditches can be planned and constructed in such a way that they will operate by gravity flow and that pumps will be unnecessary. Periodic maintenance will represent another cost factor.

DEWATERING OF LEASE

The total dewatering scheme has been broken down into two stages:

Stage I - that southwestern portion of the lease in which the mining face will be northwest-southeast

Stage II - the remainder of the lease in which the mining face will be northeast-southwest.

STAGE I

Purpose

This portion of the dewatering scheme involves dewatering of the overburden by wells. These wells will be installed when water level measurements indicate that a lowering of the water table is desirable. Map 1 shows the location of the proposed wells.

Decisions as to the necessity for installation of the wells should be made about two years before overburden removal will reach the area. Wells should be constructed and pumped if the water level in nearby piezometers indicates that the saturated portion of the overburden is greater than half of the total thickness of the overburden. For example, withdrawal wells are now justified at CH 131 and CH 518 (and probably also the Stage II auxiliary well at CH 652) because the water levels at P6 and P26 are quite high and because excavation will be taking place there within two years. Decisions on pumping at CH 127 and CH 519 will be necessary in the near future.

The decision on the necessity of certain wells should be made two years ahead of excavation so that the well can be pumped continuously for nearly that whole length of time. This should, in most instances, result in a lowering of the water level sufficient to allow overburden removal activities to continue (see

Table 6). The apparently long period of pumping for each well is designed as a safety factor due to the lack of information on aquifer characteristics. Should water levels decline rapidly at large distances from the well then pumping can be curtailed and resumed at a time when excavation is more imminent. The apparently low hydraulic conductivity values make me feel that it is better to allow a slightly longer period of pumping than to be faced with dewatering which cannot take place in the allotted time.

The proposed well locations were selected on the basis of: 1) presently existing water levels indicated the probable need for dewatering, and/or 2) a possible aquifer was noted in an exploration hole log. Again, the actual need for these wells will be determined in time through the measurements in the piezometers. The number of well locations shown on Map 1 is the result of a compromise between attempts at minimal numbers of wells and a realistic safety factor; the lack of hydrologic information makes prediction quite tenuous (see Appendix A). It is not unlikely that the needed number of wells for Stage I dewatering could double, although I feel that the proposed number is adequately realistic.

Construction

It should be borne in mind here that a pessimistic outlook could justify doubling the following figures.

Wells

Total of nine for the entire Stage I program but with a probable maximum of six in operation at any one time. Appendix B shows specifications for all wells in the lease. The screen and the casing are recoverable and reusable. Casing requirements would be about six 5-foot segments of 7 inch. Both of these are projections based on about six wells operating at any one time.

Pumps

It is felt that the pumps obtained should have a capacity of 0 and 100 gpm. Again, six at any one time would probably be sufficient. The type of pump is a cost option, the only requirement being that it is capable of delivering up to 100 gpm from depths up to 120 feet (see proposed well near CH 221) for long periods of time. These pumps are technically reusable but length of service under continuous pumping must be considered. Appendix B depicts schematically the construction and pump placement in any well used in the dewatering program.

Power Supply

The power supply for each pumping site will depend upon the type of pump selected.

Conveyance

It will be necessary to provide piping and/or ditching to convey the water out of the lease to the Athabasca River. The muskeg ditches should be suitable.

Shelters

Pumping on a year-round basis should be anticipated and so about six small shelters will be necessary to enclose the well head and allow maintenance work to procede in reasonable comfort.

STAGE II

Purpose

This portion of the dewatering procedure involves two major divisions with some minor alternatives contained within each of these. The goal of the first major division is both to cut off groundwater inflow to the lease from the southwest and removal of water from within the lease itself. The second major division involves the placement of auxilliary withdrawal wells at selected sites within the lease boundaries to further lower the water levels. These auxilliary wells will be constructed at places where they are needed based on measured water levels.

First Division

The goal of the first major division of Stage II can be accomplished to varying degrees by one of two methods. Cost analysis should be applied to both to determine the most feasible method. I will make comments as to their relative effectiveness which should be given consideration along with the cost considerations.

Main Ditch

Map 1 shows the proposed route of a major ditch along the southwestern margin of the lease and a cross section along that trend. This ditch, about 14,500 feet long, ideally should extend from the surface to the top of the ore body. Practically, however, due to the unevenness of the surface of the ore body, the ditch could be dug to an elevation of about 1020 feet at the south end and about 980 feet at the north end. This will result in a bottom slope drop of 40 feet over the total length of the ditch. The actual specifications for the size of the ditch are left up to G.C.O.S.

It seems that the obvious way to slope the bottom of the ditch is northward; with this in mind it is possible to construct an outlet for this ditch toward the northeast. This outlet is depicted on Map 1; it consists of a continuation of the ditch for about 3600 feet to the face of the escarpment at an elevation of 970 feet.

The alternative to this outlet ditch would be to pump the water out of the ditch and convey it to the Athabasca River. This would imply continuous, year-round pumping for a period of about 30 years. The cost of maintaining and pumping for this period of time should be weighed against the cost of the outlet ditch.

Implementation

Plan I

The ideal and most effective mode of implementation would be to construct the ditch all at once. If this is done, then the long-term drawdown effects of the ditch could be utilized to the utmost. The result could very well be that many of the auxilliary wells in the interior of the lease would not be necessary (Table 3). The rates of pumping under this plan (if the outlet ditch was not elected) could be expected to be 3-5 mgpd (million gallons per day) initially, dropping to about 1.0 mgpd in the long run.

Plan II

This mode of operation, which is less desirable from the technical point of view, calls for constructing the same ditch but in short segments. Under this plan it is recommended that the ditch be opened at any point for no less than four years before overburden excavation reaches that same point. This time period was arrived at by considering the drawdowns possible within the bounding hydraulic conductivities (see DEWATERING PHILOSOPHY). Thus, for instance, under a "2-year plan" initial ditch construction would call for extending it out to the limit of overburden removal six years hence (the required four years plus two more). Two years later the ditch would again be extended to the limit of overburden removal six years from that date. Thus the ditch would affect water levels adjacent to it for periods of no less than four years.

I will emphasize that I find Plan II less desirable than Plan I from a technical point of view. It is likely that more auxilliary wells will have to be constructed under Plan II than under Plan I due to the shorter time of influence of the ditch. There will also have to be continual moving of the pumping station and discharge conveyances in Plan II which would not be necessary in Plan I. Pumping rates under Plan II would obviously be much lower than in Plan I due to the shorter length of the ditch.

It is recommended that pumping from the ditch be done continuously throughout the year. During spring, summer, and fall the rates of discharge should be high enough to hold the water level at the bottom of the ditch. As freeze-up approaches, pumping rates should be reduced so as to cause water levels in the ditch to rise. If the depth of water in the ditch is kept at 10 or 15 feet all winter then pumping can take place from beneath the ice cover.

The total cessation of pumping in the main ditch is not recommended. Doing this will allow water levels to rise as water moves into the lease from the southwest. Maintaining low water levels in the ditch year round will prevent this movement and will probably reduce the number of auxilliary wells necessary.

A pump house, power supplies and conveyances for discharged water will be necessary if the outlet ditch is not opted for.

Row of Wells

An alternative to the construction of the main ditch is a row of pumping wells following the same trend as that indicated on Map 1 for the main ditch. Theoretically, to simulate the effects on water levels by a ditch these wells would have be be placed immediately adjacent to each other. Since this is not practically realistic, it is necessary to have some distance separating each well; of course, as this distance increases the row of wells becomes less similar to a ditch.

It appears, from theoretical distance-drawdown equations (see Table 6), that a spacing between wells of 300 to 400 feet might be adequate to simulate the effects of the ditch to a reasonable degree. These wells should be pumped year round in order to exert the greatest influence at the greatest distance and simulate the ditch as closely as possible.

Implementation

As with the main ditch the line-of-wells alternative can be implemented on any magnitude between a maximum (Plan I) and a minimum (Plan II).

Plan I

The ideal and most effective mode of implementation is to construct the line of wells all at once and commence continuous pumping immediately. This would entail construction of about 40 wells at spacings gradually increasing from 300 feet in the south to 400 feet in the north. The implementation of this plan under continuous pumping could probably make many of the auxilliary wells unnecessary.

Wells - Construction as described in Appendix B. The approximately 40 wells should require about 2400 feet of 12 inch hole and casing plus 5-foot screen segments for each well.

<u>Pumps</u> - About 40 pumps of 0-100 gpm capacity. The type of pump is not specified; it must be capable of delivering water from the top of the ore body to the surface, an average distance of 60 feet.

<u>Power supply</u> - The type of power at each site will depend on the type of pump selected. The amount of power depends on the rate of pumping and the necessary lift but might average 50 gpm and 60 feet, respectively.

<u>Conveyance</u> - It will be necessary to provide piping and/or ditching to convey the water out of the lease to the Athabasca River. Muskeg ditches would be suitable.

<u>Shelters</u> - Pumping on a year-round basis should be anticipated and thus shelters for each well head will be required.

Plan II

This mode of implementation is less desirable from the technical point of view. It calls for constructing the wells along the trend of the ditch in short segments. The criteria here is that a well be constructed and pumped for a period of no less than four years before overburden removal reaches that point. The four-year time criteria was arrived at through consideration of the bounding hydraulic conductivities of Table 6. This plan has the advantages of allowing re-use of some of the equipment and of lower power cost to the system at any one time. Another advantage is that if the recommended well spacing should be incorrect, it can be adjusted with additional increment of wells. A disadvantage is that installation of all of the auxilliary wells will probably be necessary to dewater central portions of the lease with the auxilliary wells.

Equipment and construction needs can be evaluated by considering the 4-year criteria, the rate of advance of overburden removal, and an approximate

300-foot spacing between wells. Appendix B shows the suggested well construction. Equipment specifications are similar to those under Plan I.

Evaluation

As has been obvious in the preceding discussion of the first division of Stage II, I feel that technically the best of the alternatives is the construction of the complete main ditch and outlet. This allows for:

1. long drainage of the materials of low hydraulic conductivity in the northern portion of the lease;
2. minimal maintenance problems as contrasted with wells;
3. no pumping costs;
4. year-round low water levels in the ditch;
5. effective elimination of groundwater flow into the lease from the southwest.

The alternatives to this ditch are presented so that a cost analysis can be made to determine the actual economic weight of each.

Second Division

The second major division of Stage II of the program involves "auxilliary" dewatering. The purpose of the auxilliary scheme is to dewater isolated "pockets" of water within the lease. This division is designed to be used in addition to any of the alternatives chosen in the first division. This division also has two alternative schemes within it which can be selected on economic grounds.

Auxilliary Wells

Map 1 shows the projected location of auxilliary wells. These sites were selected on the basis of one or more of the following: 1) probable need to dewater depressions on the tar sand surface, 2) existence of a possible aquifer, 3) distance from the main ditch. The need for these wells will be determined from measurements in the piezometer network. If it appears that 70 per cent of the overburden section at an auxilliary well location is still saturated two years before excavation is to reach that point, then a dewatering well should be constructed at that site. Pumping can then take place for the major portion of those two years. It may not be necessary to pump throughout the winter if the entire main ditch or line of wells is in place and operative all year round. This is assuming that the main ditch or line of wells is installed soon and has been operating for several years before Stage II overburden removal begins.

It is very likely, provided the main ditch or line of wells is fully completed and the muskeg ditching functional, that many of the auxilliary wells of the interior of the lease will not be necessary. It is likely that those wells west of the ditch trend will be necessary in any case because of the sustaining effect on water levels caused by groundwater flow from the west.

Implementation

Wells - Appendix B shows a schematic drawing of well construction and pump location. There are 37 proposed wells on Plate 1 with a total footage of about 2300 feet. (This figure could decline significantly depending on the option selected in the First Division and its mode of implementation.)

It appears that no more than about eight individual wells will be needed at any one time. This would mean that about 500 feet of seven inch pipe and eight 5-foot screens should satisfy needs.

Pumps - Pumps should have a capacity between 0 and 100 gpm and be of a type which can sustain long-term pumping. The number needed at any one time should also be eight and they are technically reusable; however, the length of service under continuous pumping will determine the total number needed for the entire dewatering period.

Power supply - The type of power will depend on the pump selected. The amount of power required at any instant should be based on about eight pumping wells, drawing water at about 50 gpm from an average depth of 60 feet.

Conveyance - Piping and/or ditches will be needed to convey the discharged water to the Athabasca River. The muskeg ditches may be suitable for this purpose.

Shelters - As noted previously, by selecting certain options of the First Division of Stage II, it will not be necessary to pump these wells all year round. In this case, well-head shelters would not be necessary. Under other options pumping all year round may be necessary and shelters would be required.

Auxilliary Ditch

Map 2 shows the traces of two deep ditches. The ditch on the left is similar in location and purpose to the main ditch but slightly west of its location. The discussion pertinent to the main ditch applies to this ditch.

The ditch on the right serves the same purpose as the Stage II auxilliary wells, namely to dewater the interior of the lease.

The construction of outlet ditches has been indicated on Map 2. These are optional, but their cost should be weighed against the long-term costs of transferring water by pumping.

This method of dewatering the overburden of the lease will most assuredly be successful provided that the western ditch is kept nearly dewatered all year round. I base this statement on the water levels in Table 5 calculated from the maximum and minimum expected hydraulic conductivities of overburden materials. The eastern ditch would probably not have to be pumped year round if the system is implemented under Plan I. Under Plan II it is likely that pumping will have to be maintained in both ditches year round.

As I stated above, this method will do the best dewatering from a technical point of view. There is the possibility, however, that it would be more effective than is warranted. It is possible that the main ditch, if fully constructed at once and kept dewatered year round, would cause adequate lowering of water levels. The auxilliary wells probably provide a lower cost alternative to construction of the eastern ditch. Therefore, from a technical point of view, I do not recommend this two-ditch approach. It may be, however, that total cost considerations of the other methods make this approach more desirable. I do feel that this method (two ditches) will result in the most complete and effective dewatering of the overburden.

Implementation

These ditches can be constructed in any manner ranging between Plan I and Plan II outlined previously for the main ditch.

CONCLUSIONS

Existing hydrogeologic information on the overburden of the G.C.O.S. lease is inadequate with respect to the determination of the distribution of hydraulic conductivity and infiltration rates. This has led to the necessity for a maxima-minima approach to removal of water from the overburden. Water levels and probable pumping rates from ditches are calculated for hydraulic conductivities of 200 gpd/sq ft and 6 gpd/sq ft and for possible infiltration rates of 0.1, 0.3, 0.5, and 0.7 cubic ft/year/sq ft. It is expected that the actual water levels and pumping rates will fall somewhere between the extremes represented by the values of hydraulic conductivity utilized in the calculations.

The geology of the overburden is well known. It varies from mostly sands and gravels in the southern third of the lease to mostly clayey till with some inter-tonguing sands in the northern two thirds. There is a major sand area at the northern lease edge along the escarpment face. It is expected that the average hydraulic conductivity of the sediments is higher in the sandy areas and much lower in the clayey portions of the lease.

A piezometer network is a necessary part of the proposed dewatering scheme. These wells can be constructed inexpensively from small-diameter pipe and sandpoints. Freezing in winter can be eliminated by adding a half gallon of heavy oil to each piezometer.

Drainage of the muskeg via shallow ditches is necessary to reduce infiltration of precipitation. This in itself could cause a significant lowering of the water table.

Various schemes for removing water from the overburden are possible and should be subjected to cost analysis.

OUTLINE FOR COST ESTIMATION

It must be remembered that I did not make any financial analysis of the recommended program to evaluate any alternative costs involved with stated options. I present the following outline in order to facilitate a cost analysis of all factors.

I. Muskeg dewatering

  A. Specifications

    1. about 900-foot spacing between ditches

    2. about 2-3 feet deep

  B. Construction

    1. small dragline

    2. specialized equipment

II. Piezometer network

  A. Specifications

    1. New -
      a. 1 1/4 inch iron pipe — 450 feet
      b. 1 1/4 inch screened sandpoint — 5
      c. small-diameter drill hole — 430 feet

    2. Rejuvenation of old —
      a. 1 1/4 inch iron pipe — 810 feet
      b. 1 1/4 inch screened sandpoints — 10
      c. small-diameter drill hole – 790 feet

  B. Monthly monitoring and recording of data

    1. Labor — 12 man-days per year

III. Stage I - Wells

  A. Specifications — (4 or 5 wells at any one time)

    1. Gravel-packed, 12-inch drill hole — 480 feet (total for Stage I)
      a. see Appendix B

    2. 7-inch steel casing — 300 feet
      a. reusable

    3. screens — 5, 5-foot segments (7 inch diameter)
      a. reusable

B. Pumps (0-100 gpm capacity)

1. 4 or 5 at any one time
2. total number depends on expected length of service
3. maintenance

C. Power

1. delivery to site
2. two years continuous pumping at each site

D. Conveyance

1. piping
2. and/or ditches

E. Shelters

1. 4 or 5 at any one time
2. reusable

F. Supervision — labor

1. manual water level observation daily
2. or self-regulating device as outlined in Appendix B

IV. Stage II

A. First Division

1. Main ditch (optional with IV-A-2)

a. Specifications
   i. total length - 14,500 feet
   ii. average depth — 60 feet
   iii. side slope, bottom width — base on G.C.O.S. experience with existing ditches

b. Pumping water removal (optional with IV-A-1-c)
   i. initially — 3-5 mgd year round
   ii. later — 1 mgd maximum year round
   iii. power to pumps
   iv. water conveyance — to Athabasca River
   v. pump house

c. Outlet ditch (optional with IV-A-1-b)
   i. total length — 3600 feet
   ii. average depth — 30 feet
   iii. side slope, bottom width — base on G.C.O.S. experience with existing ditches

d. Construction procedure
   i. Plan I — whole ditch at once
   ii. Plan II — short segments; no less than 4 years ahead of excavation; must use IV-A-1-b
   iii. above two represent maxima and minima

2. Row of wells (optional with IV-A-1)

a. Well specifications: (see Appendix B)
   i. about 300-400 foot spacing - 40 wells (average depth of 65 feet)
   ii. gravel-packed 12-inch drill hole - about 2600 feet
   iii. well screen — 40 5-foot segments (7 inch)

b. Pumping requirements
   i. 0-100 gpm capacity
   ii. continuous service
   iii. maintenance
   iv. year-round operation

c. Power
   i. delivery to site
   ii. year-round pumping

d. Shelter

e. Conveyance to Athabasca River
   i. piping
   ii. or ditches

f. Supervision
   i. one operator all shifts
   ii. or — self-regulating devices with minimal supervision

g. Construction procedure
   i. Plan I - whole line at once
   ii. Plan II - short segments; no less than 4 years ahead of excavation
   iii. above two represent maxima and minima
   iv. implications of shorter segments discussed in text

B. Second Division

1. Auxilliary wells — ad hoc basis (optional with IV-B-2)

a. Specifications
   i. locations — as on Plate 1
   ii. number — 37
   iii. gravel-packed, 12-inch drill hole — 2270 feet total (see Appendix B)
   iv. pipe — 7 inch, 500 feet (reusable)
   v. well screens — 10 5-foot sections (7 inch) (reusable)

b. Implementation
   i. ad hoc basis from piezometer network information
   ii. decide if needed 2 years before excavation
   iii. about 6 at any one time

c. Pumps
   i. operation for 2 summer seasons probable under Plan I
   ii. continuous operation year-round under Plan II
   iii. 0-100 gpm pump capacity
   iv. maintenance
   v. length of service
d. Power
   i. delivery to sites
   ii. amount dependent on plan selected in First Division
e. Conveyance
   i. piping
   ii. and/or ditches to Athabasca River
f. Shelters
   i. about 10 at any one time
g. Supervision
   i. manual water-level observations daily
   ii. or — self-regulating device as outlined in Appendix B

2. Two Ditches (Plate 2) (optional with IV-B-1)

a. Specifications
   i. east ditch — 11,600 feet long
   ii. west ditch — 16,800 feet long
   iii. average depth -- about 60 feet
   iv. side slope, bottom width — base on G.C.O.S. experience with existing ditches
b. Pumping water removal (optional with IV-B-2-c)
   i. initially 1-5 mgd
   ii. later 1-2 mgd
   iii. year-round pumping in west ditch
   iv. summer season in east ditch
   v. power to pumps
   vi. water conveyance — to Athabasca River
   vii. pump houses
c. Outlet ditch (optional with IV-B-2-b)
   i. east ditch — 1800 feet
   ii. west ditch — 4000 feet
   iii. average depth — about 30 feet
   iv. side slope, bottom width — base on G.C.O.S. experience with existing ditches
d. Construction procedure
   i. Plan I — both ditches at once
   ii. Plan II — short segments; no less than 4 years ahead of excavation; must use IV-B-1-b
   iii. above represent maxima and minima

REFERENCES CITED

Canada Department of Transport, Meteorological Branch (1967): Temperature and precipitation tables for Prairie Provinces; Vol. III.

Clark, K. A. (1960): Permeabilities of Athabasca oil sands; Can. Mining and Metallurgical Bull., Vol. 53, No. 576, pp. 239-244.

Davis, S. N. and R. J. DeWiest (1966): Hydrogeology; John Wiley & Sons, Inc., 463 pages.

Glover, R. E. (1964): Ground-water movement; U.S. Dept. of the Interior; Bureau of Reclamation, Engineering Monograph No. 31, 76 pages.

Kahil, A. (1968a): Memo on the effect of the sump on the dewatering of the overburden in G.C.O.S. lease 86; Unpublished Res. Coun. Alberta Internal Rept., July, 4 pages.

——— (1968b): Interim report on the feasibility study for dewatering the overburden in G.C.O.S. lease 86; Unpublished Res. Coun. Alberta Internal Rept., November, 39 pages.

——— (1969): Second interim report on the feasibility study for dewatering the overburden in G.C.O.S. lease 86; Unpublished Res. Coun. Alberta Internal Rept., November, 14 pages.

Linckens, W. M. J. (1965): Final report on the overburden - lease 4, Athabasca River area, Alberta-Canada; Unpublished report for Great Canadian Oil Sands Ltd., 46 pages.

Luthin, J. N. (1966): Drainage Engineering; John Wiley & Sons, Inc., 250 pages.

Thornthwaite, C. W. and J. R. Mather (1957): Instructions and tables for computing potential evapotranspiration and the water balance; Drexel Institute of Technology, Laboratory of Climatology, Publications in Climatology, Vol. X, No. 3.

Todd, D. K. (1959): Ground Water Hydrology; John Wiley & Sons, Inc., 336 pages.

Walton, W. C. (1962): Selected analytical methods for well and aquifer evaluation; Ill. State Water Surv. Bull. 49, 81 pages.

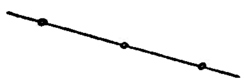
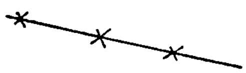
APPENDIX A

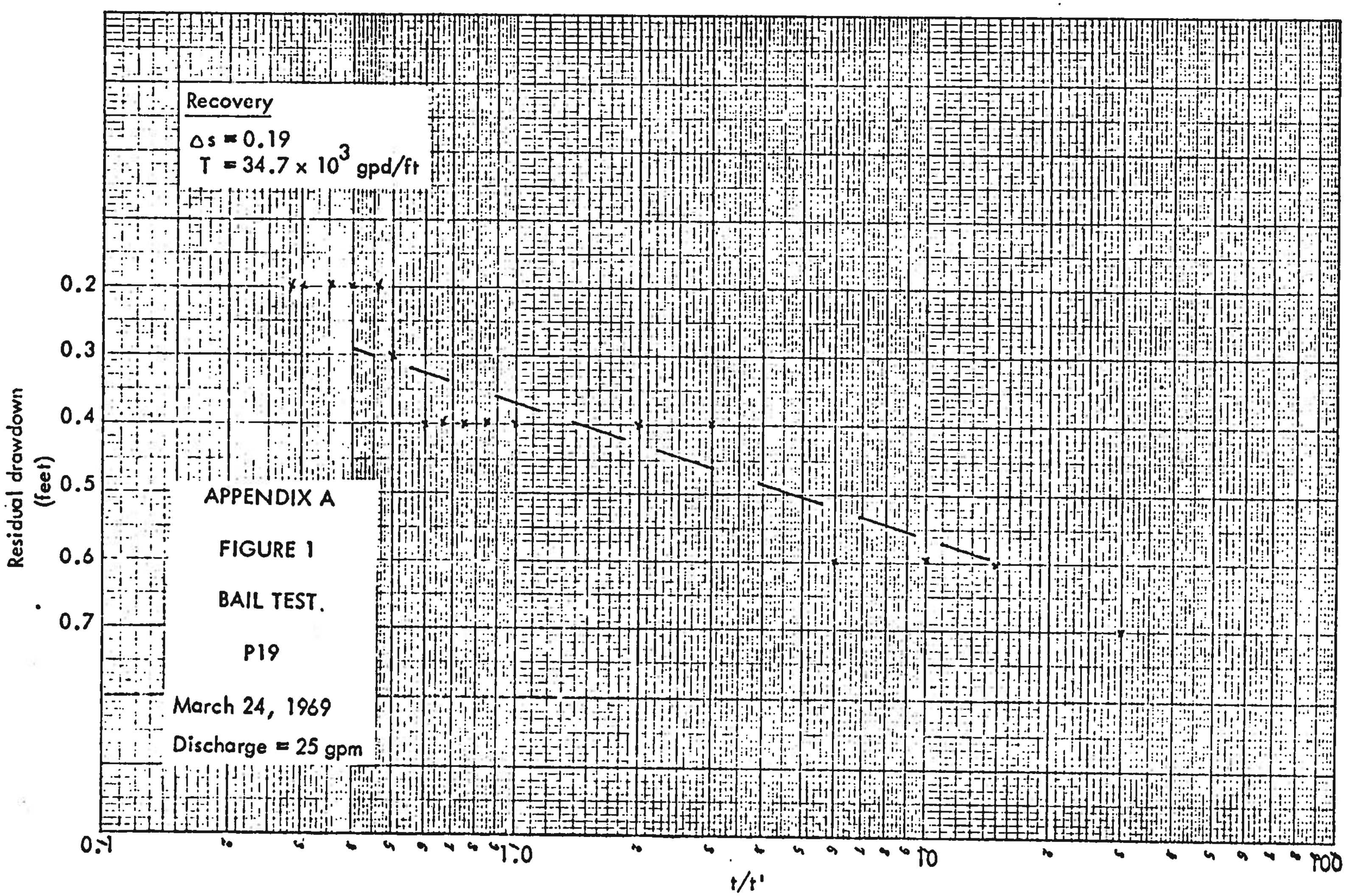
DATA AND INTERPRETATIONS OF AQUIFER PERFORMANCE TESTS

This appendix presents data and interpretations of aquifer performance tests conducted at the G.C.O.S. lease between 1967 and 1970. Aquifer transmissibility is calculated by the Jacob method.

Data from testing of P50 and P51 were not plotted due to the shortness of the tests.

Key to Following Figures

	Data points during pumping or bailing phase and straight line fit to same.
	Data points during recovery phase and straight line fit to same.
t/t'	Ratio of time since pumping commenced (t) to time since pumping stopped.
Adjusted residual drawdown or adjusted drawdown	Drawdown or residual drawdown data has been adjusted for decrease in saturated thickness by the method of Jacob as noted in Walton (1962).
$T = \frac{264Q}{\Delta s}$	Transmissibility by modified Theis equation.



APPENDIX A

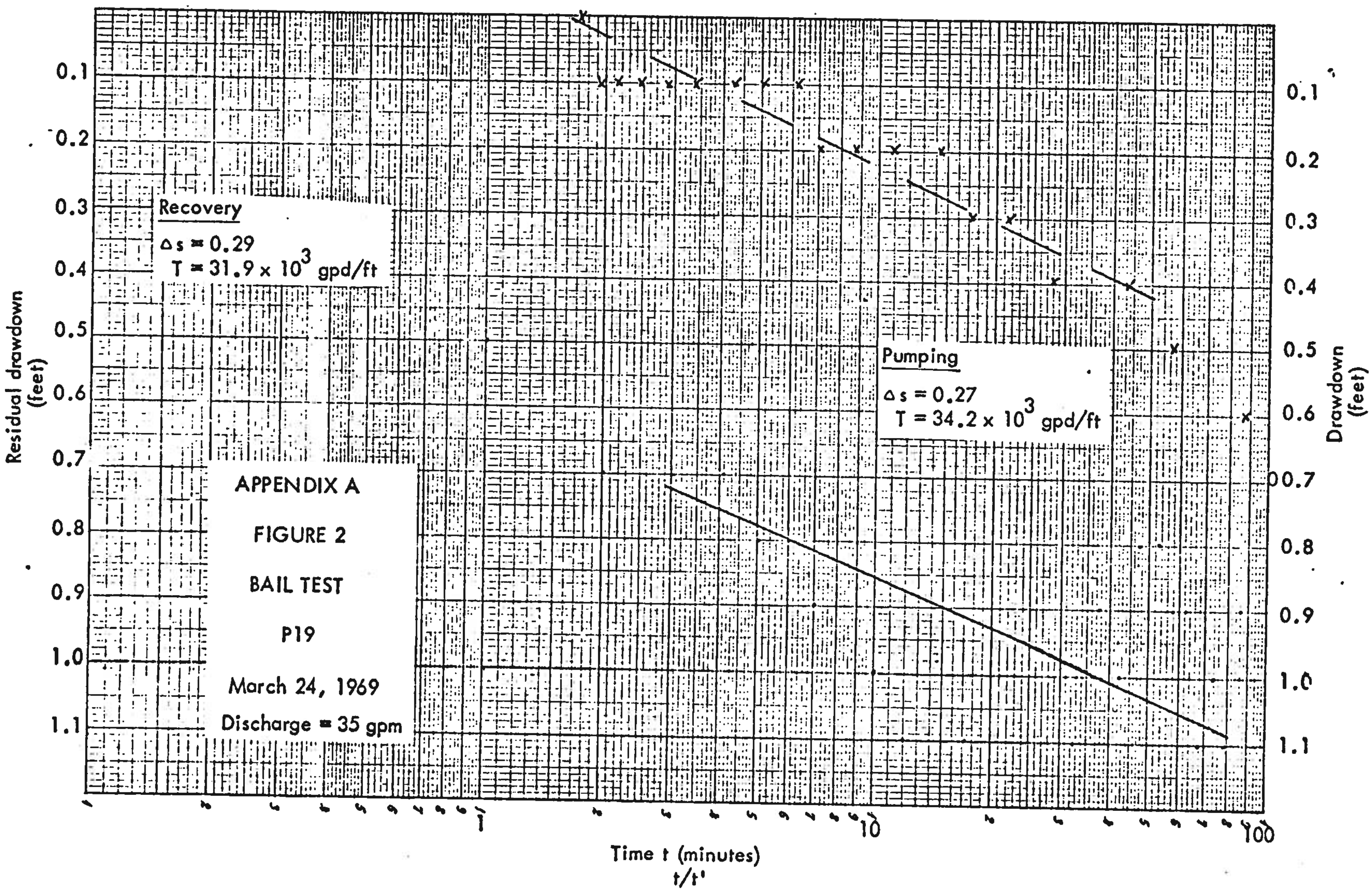
FIGURE 1

BAIL TEST.

P19

March 24, 1969

Discharge = 25 gpm



APPENDIX A

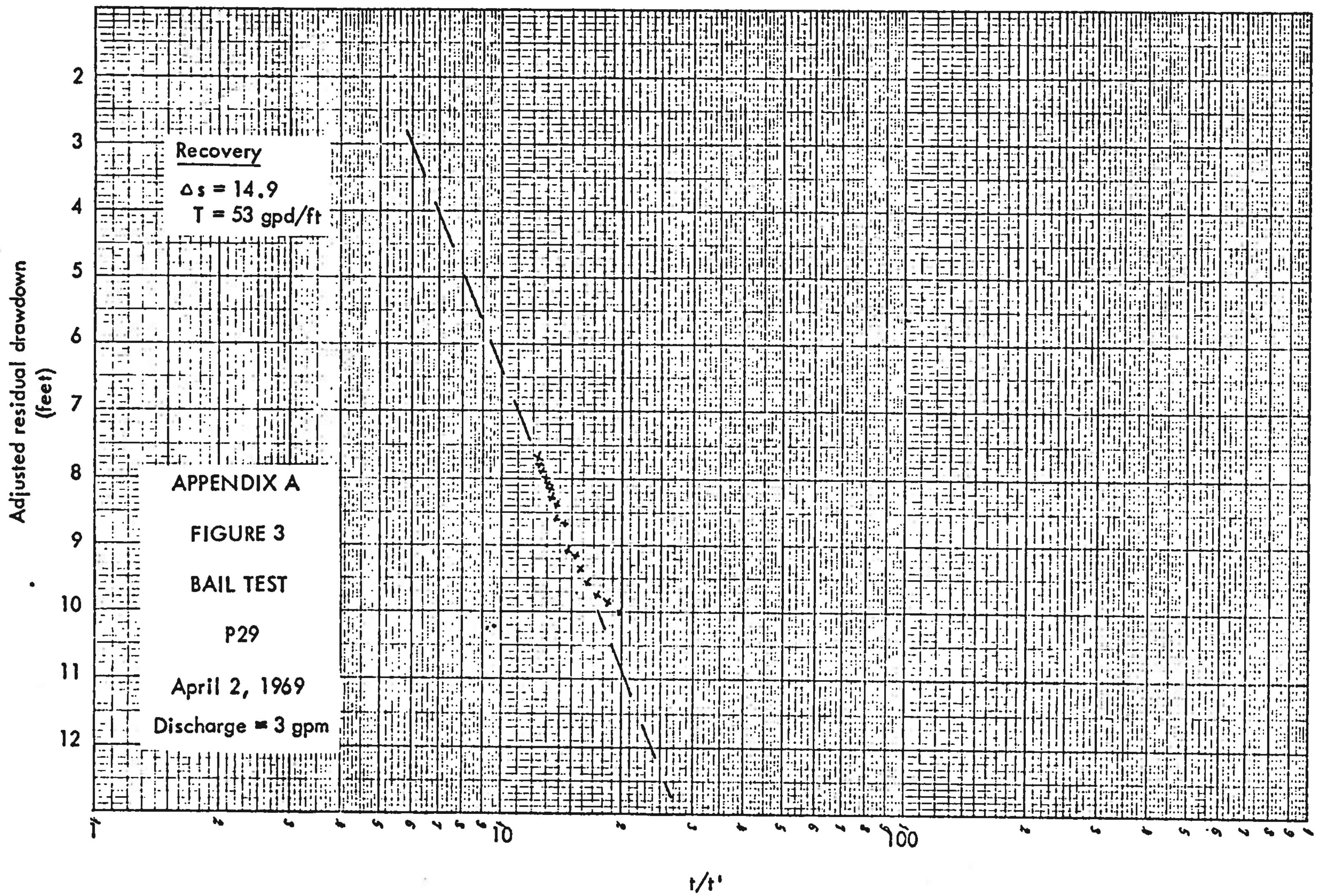
FIGURE 2

BAIL TEST

P19

March 24, 1969

Discharge = 35 gpm



APPENDIX A

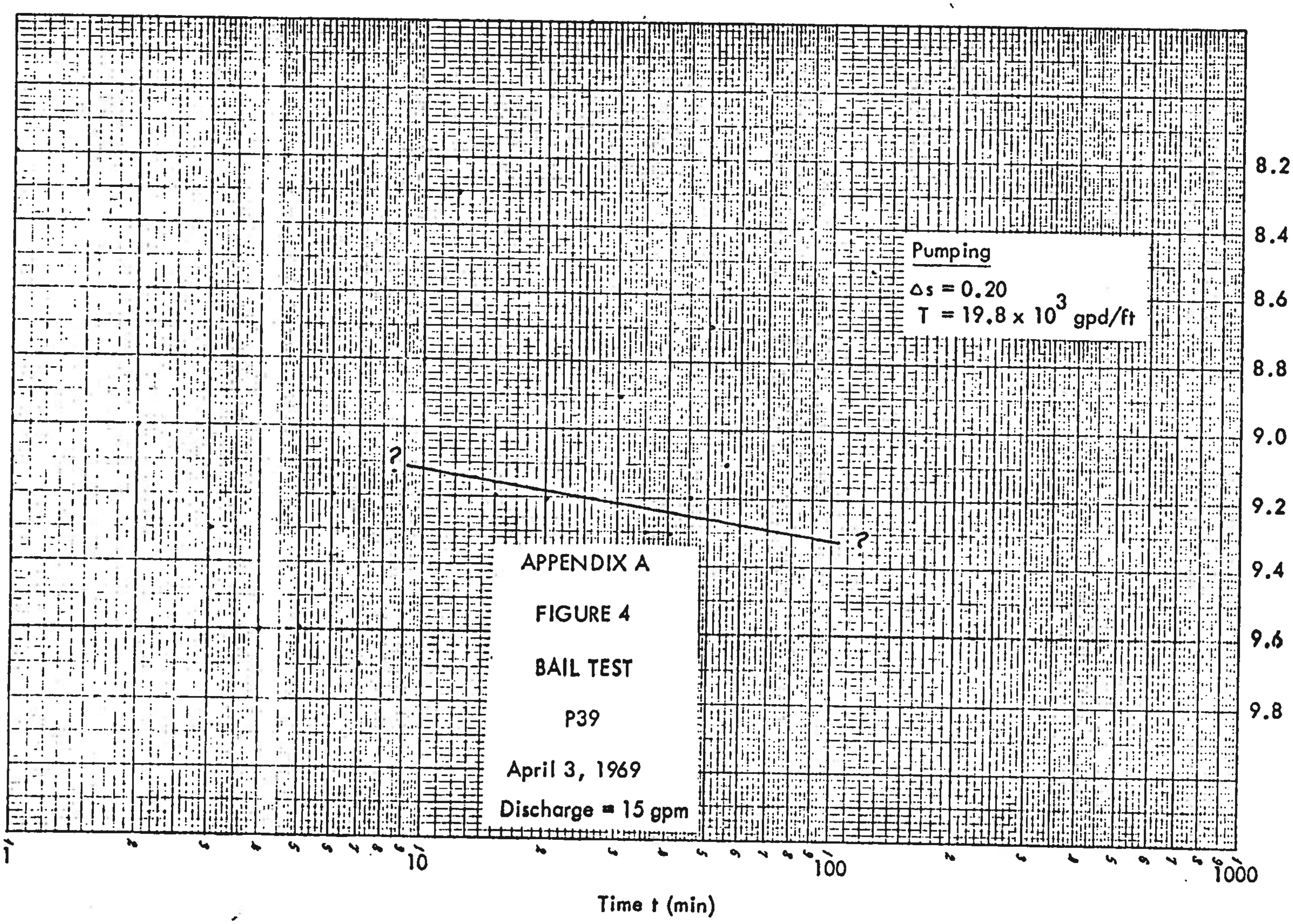
FIGURE 3

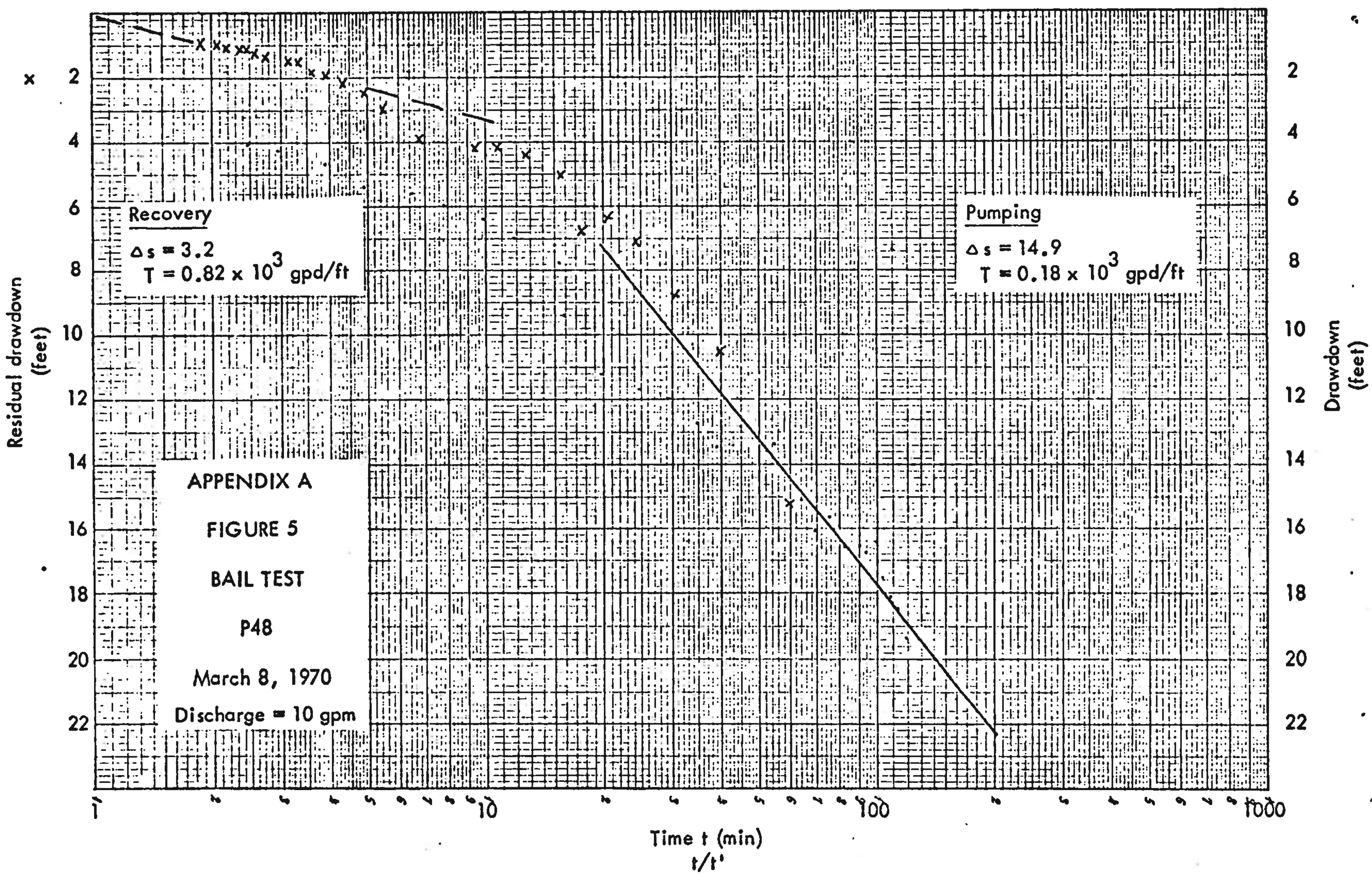
BAIL TEST

P29

April 2, 1969

Discharge = 3 gpm


Pumping
Δs = 0.20
T = 19.8 x 10^3 gpd/ft
APPENDIX A
FIGURE 4
BAIL TEST
P39
April 3, 1969
Discharge = 15 gpm
8.2
8.4
8.6
8.8
9.0
9.2
9.4
9.6
9.8
Drawdown (feet)
1
10
100
1000
Time t (min)



APPENDIX A

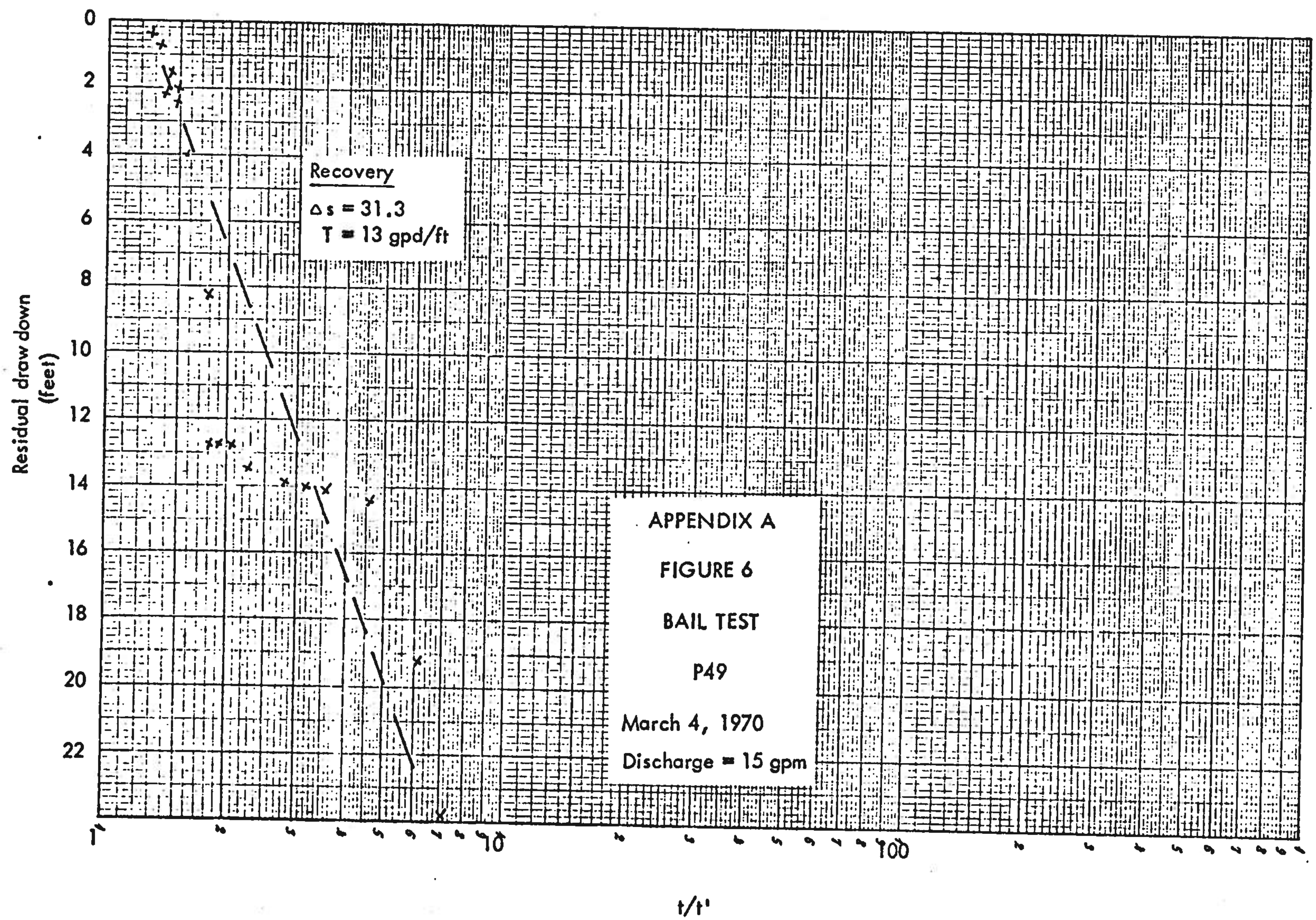
FIGURE 5

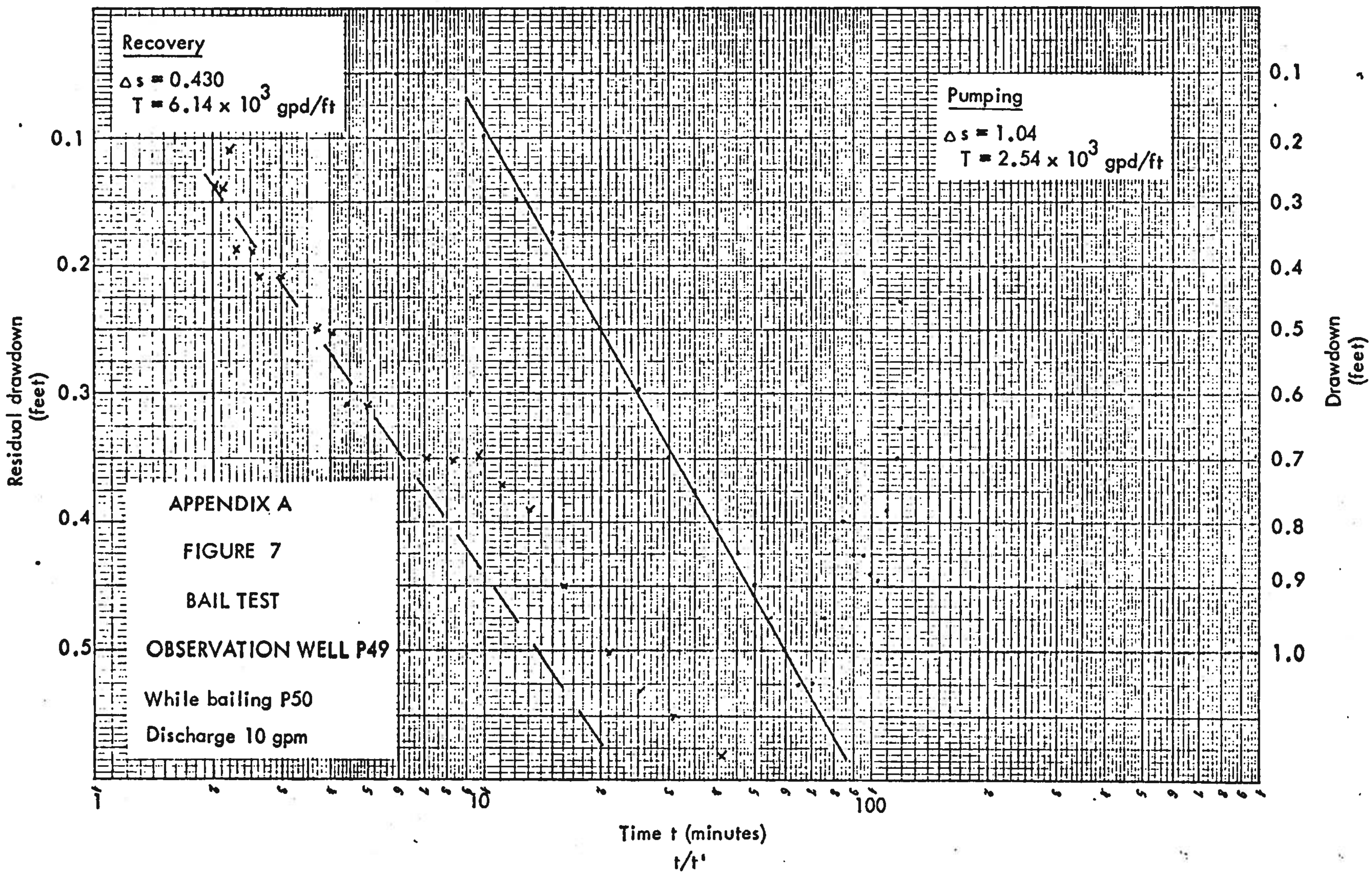
BAIL TEST

P48

March 8, 1970

Discharge = 10 gpm


Recovery
Δs = 31.3
T = 13 gpd/ft
APPENDIX A
FIGURE 6
BAIL TEST
P49
March 4, 1970
Discharge = 15 gpm
Residual draw down (feet)
0
2
4
6
8
10
12
14
16
18
20
22
1
10
100
t/t'



APPENDIX A

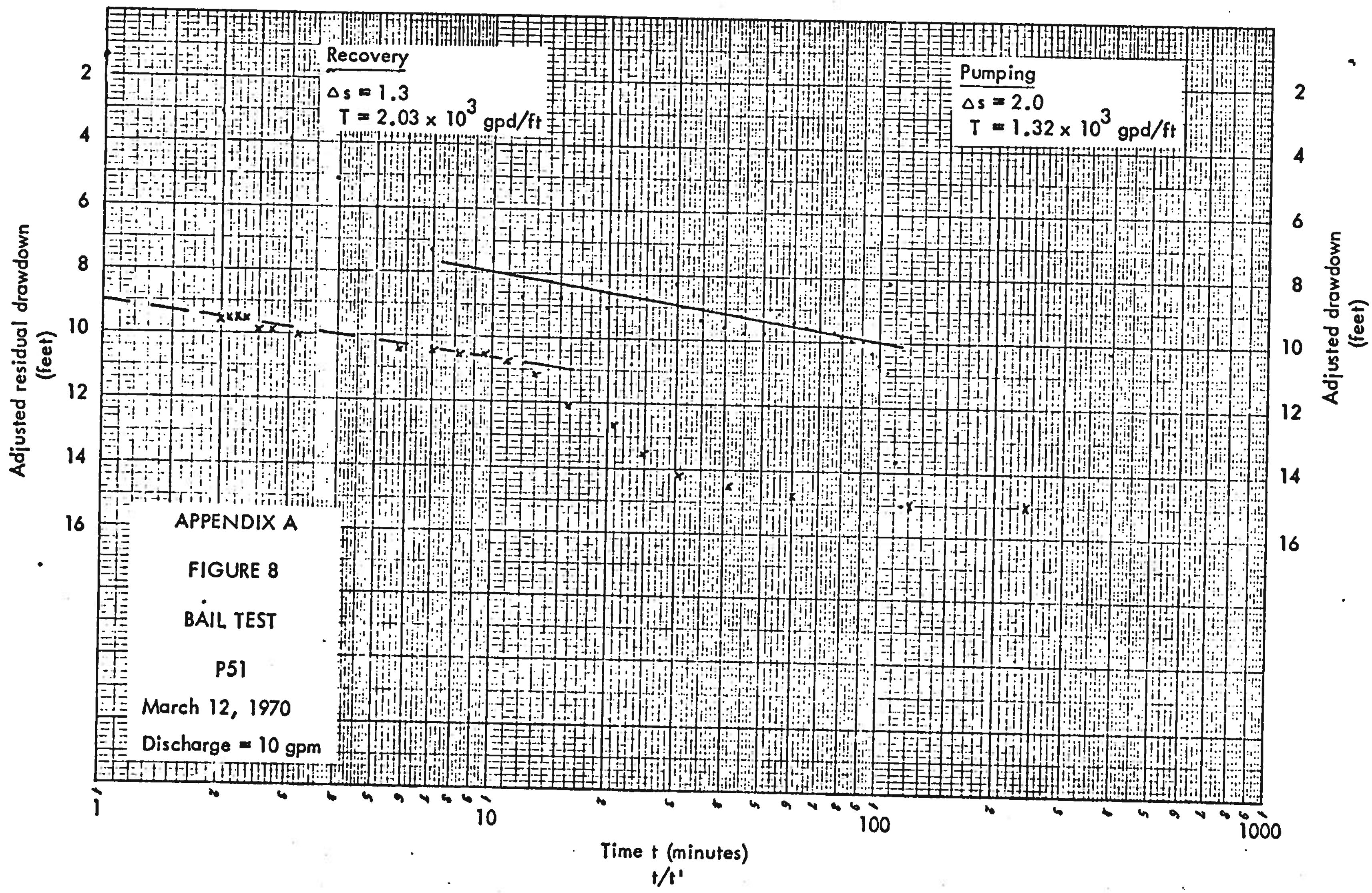
FIGURE 7

BAIL TEST

OBSERVATION WELL P49

While bailing P50

Discharge 10 gpm



APPENDIX A

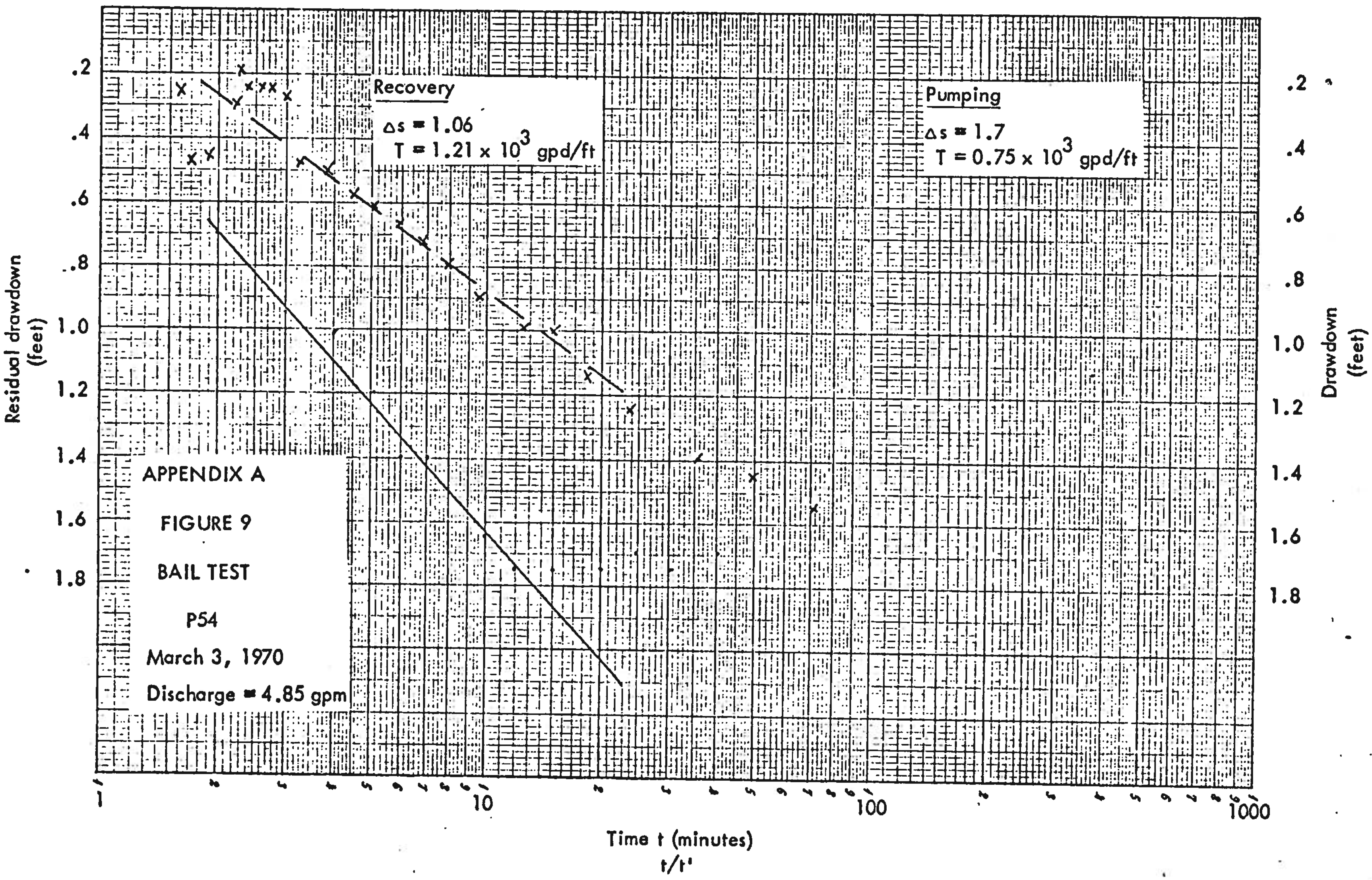
FIGURE 8

BAIL TEST

P51

March 12, 1970

Discharge = 10 gpm



APPENDIX A

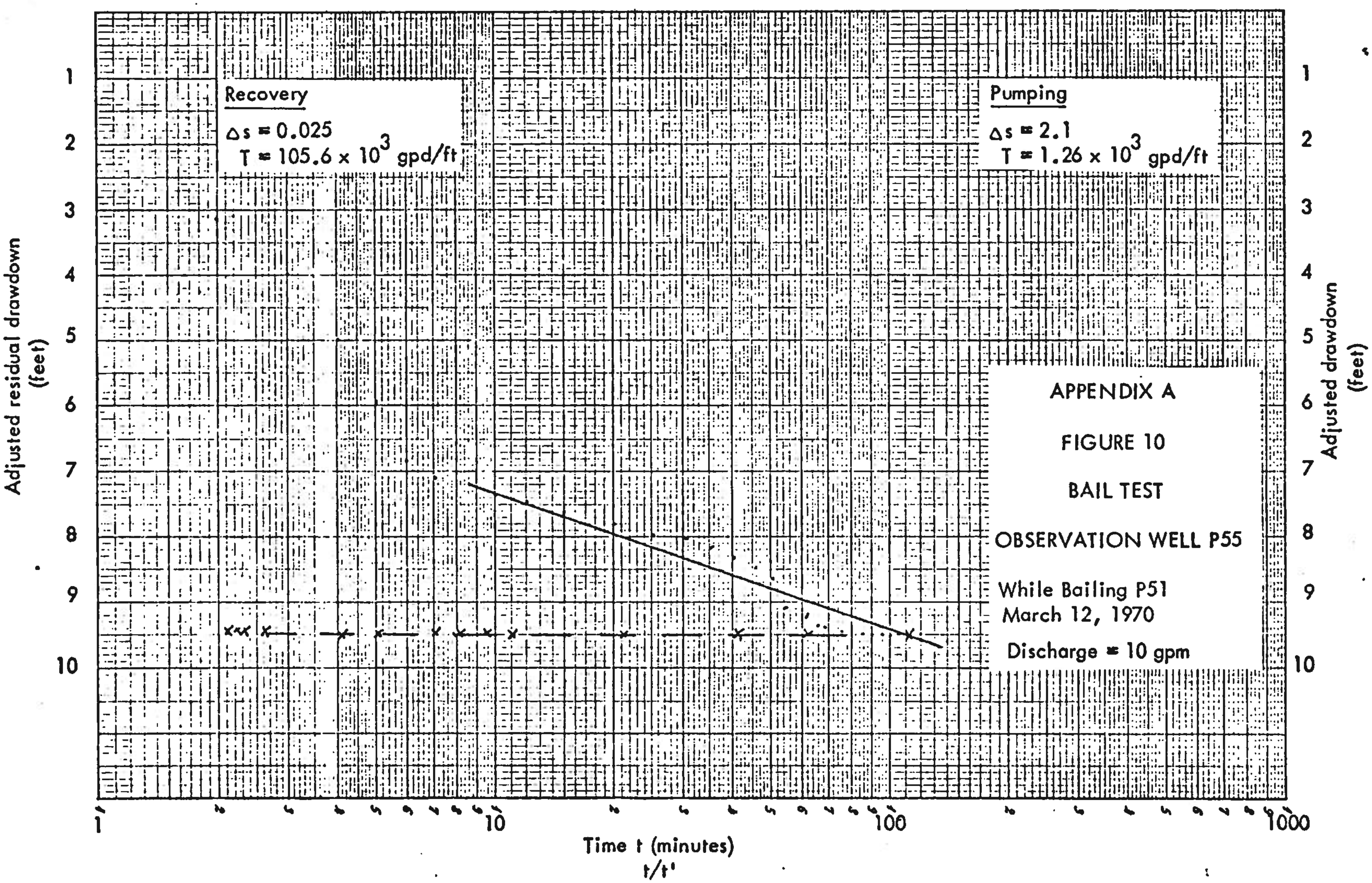
FIGURE 9

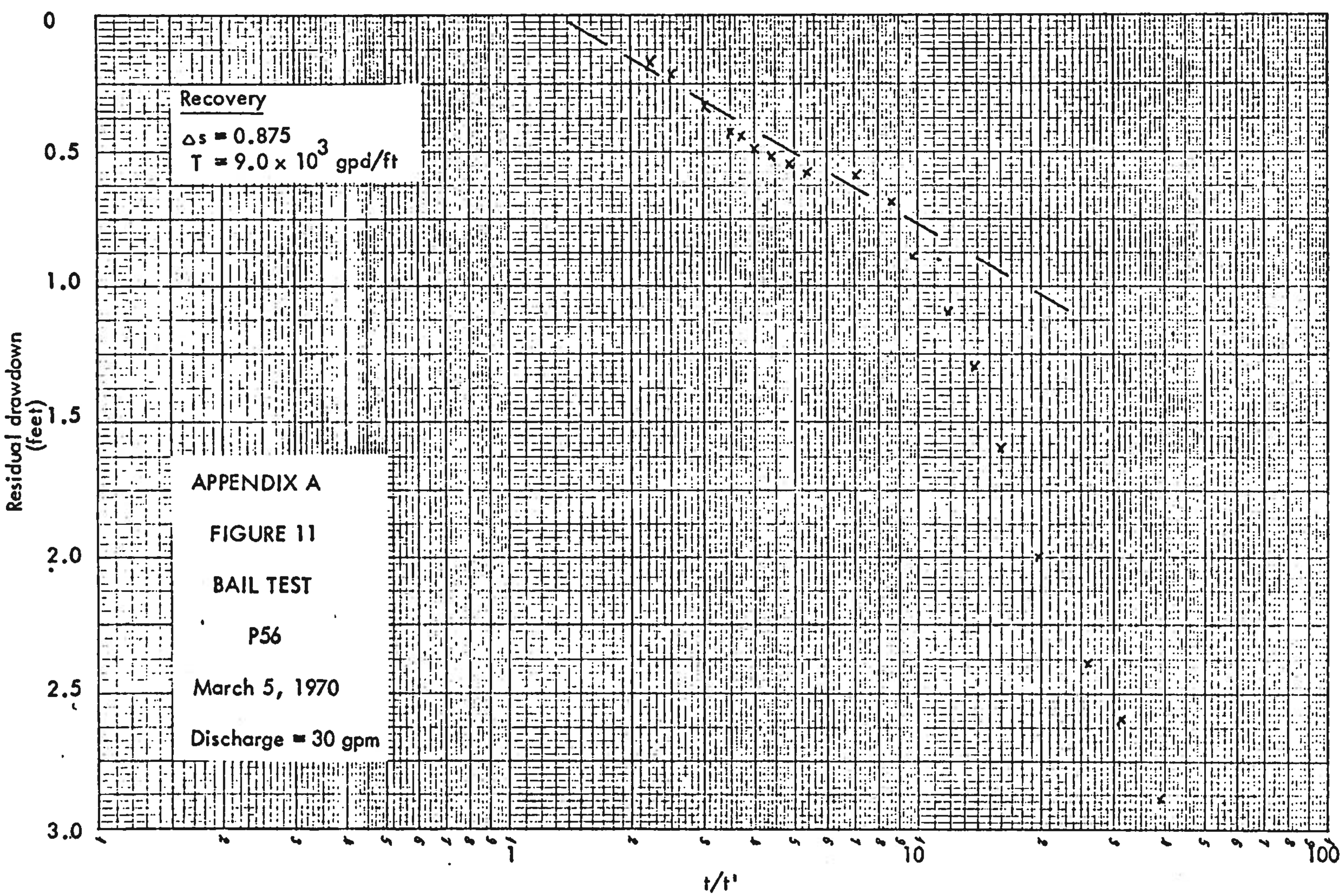
BAIL TEST

P54

March 3, 1970

Discharge = 4.85 gpm


Recovery
Δs = 0.025
T = 105.6 x 10^3 gpd/ft
Pumping
Δs = 2.1
T = 1.26 x 10^3 gpd/ft
APPENDIX A
FIGURE 10
BAIL TEST
OBSERVATION WELL P55
While Bailing P51
March 12, 1970
Discharge = 10 gpm
Adjusted residual drawdown (feet)
Adjusted drawdown (feet)
1
2
3
4
5
6
7
8
9
10
1
10
100
1000
Time t (minutes)
t/t'



APPENDIX A

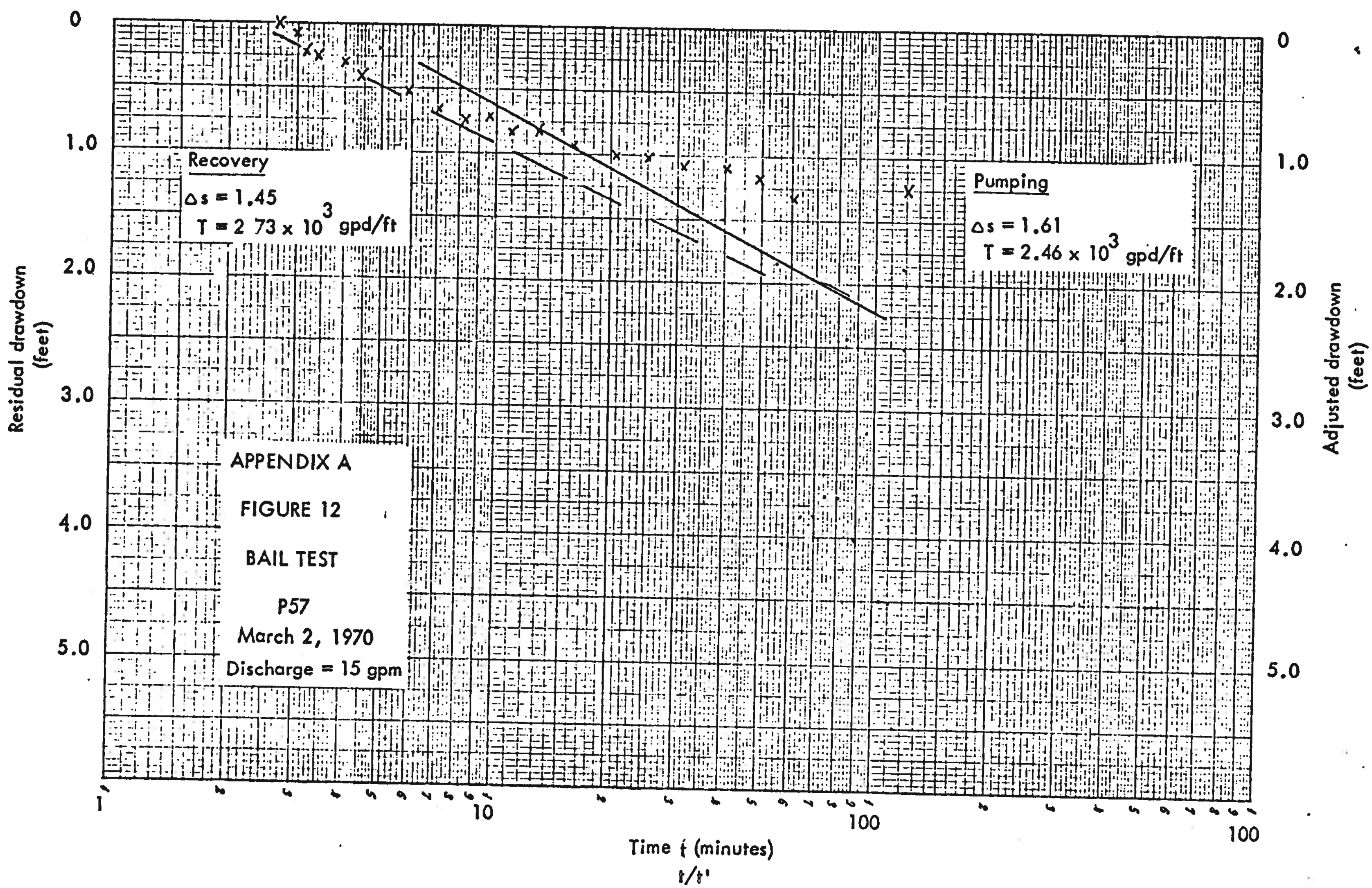
FIGURE 11

BAIL TEST

P56

March 5, 1970

Discharge = 30 gpm



APPENDIX A

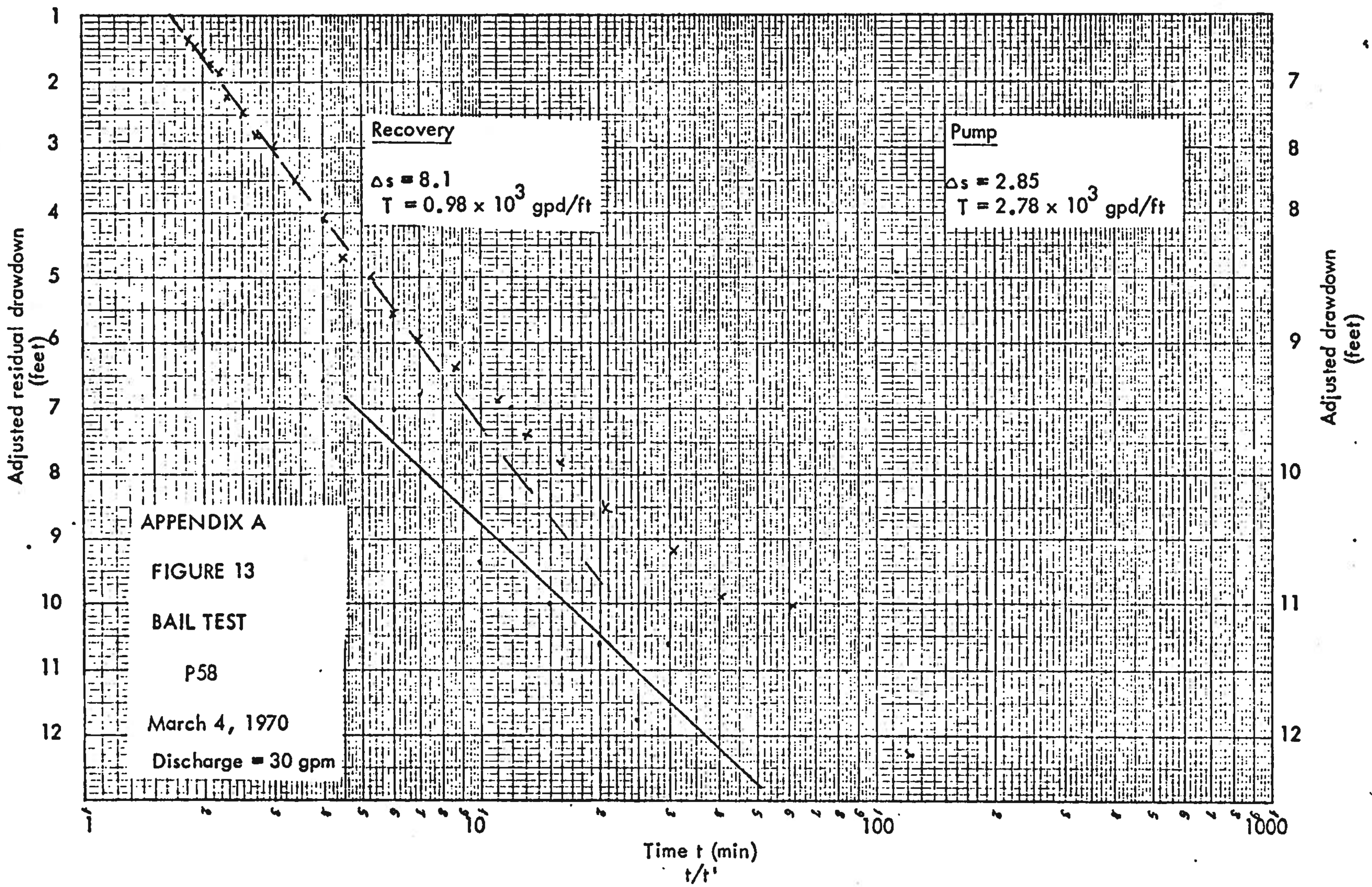
FIGURE 12

BAIL TEST

P57

March 2, 1970

Discharge = 15 gpm



APPENDIX A

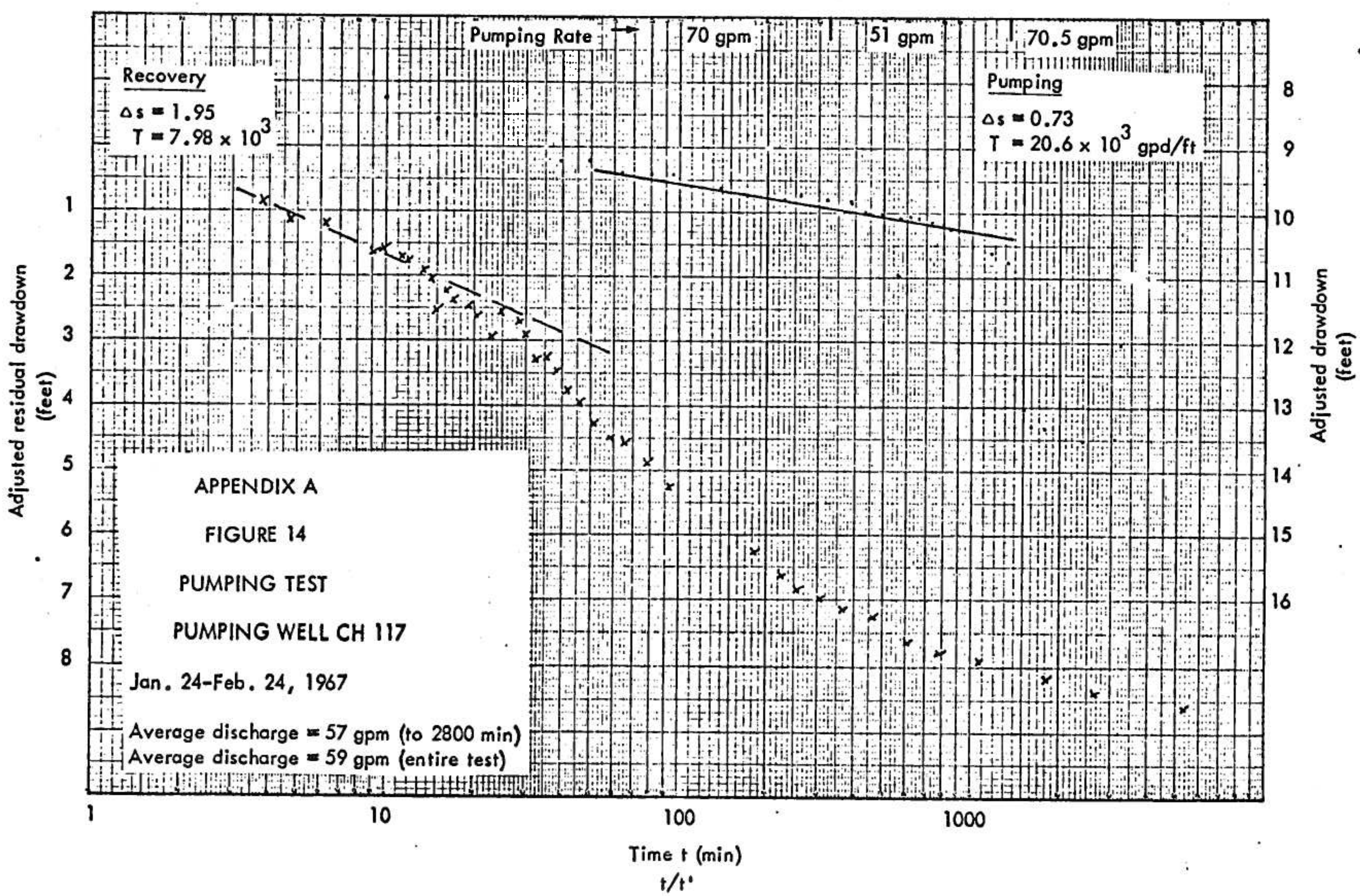
FIGURE 13

BAIL TEST

P58

March 4, 1970

Discharge = 30 gpm



APPENDIX A

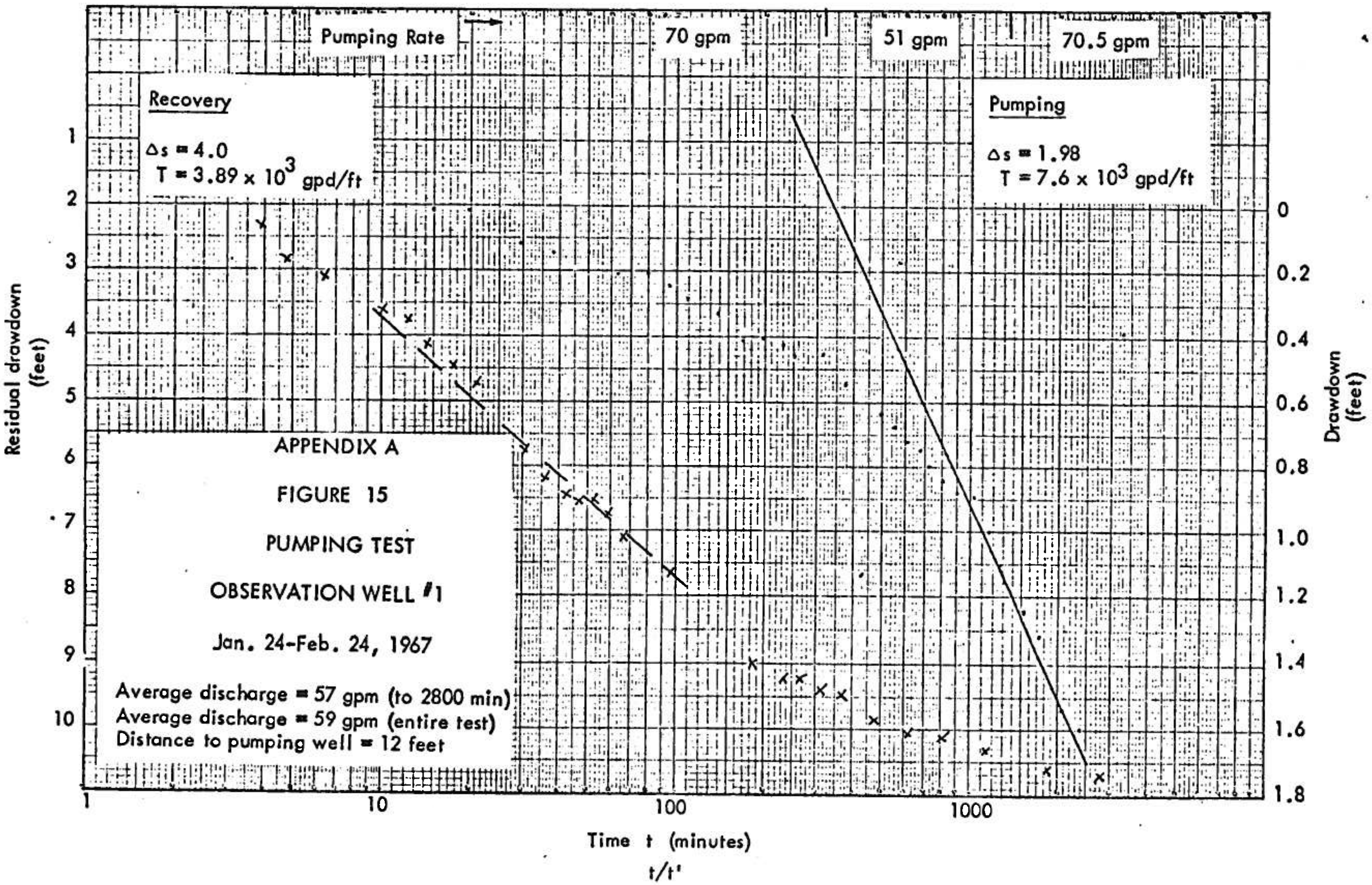
FIGURE 14

PUMPING TEST

PUMPING WELL CH 117

Jan. 24-Feb. 24, 1967

Average discharge = 57 gpm (to 2800 min)
Average discharge = 59 gpm (entire test)



APPENDIX A

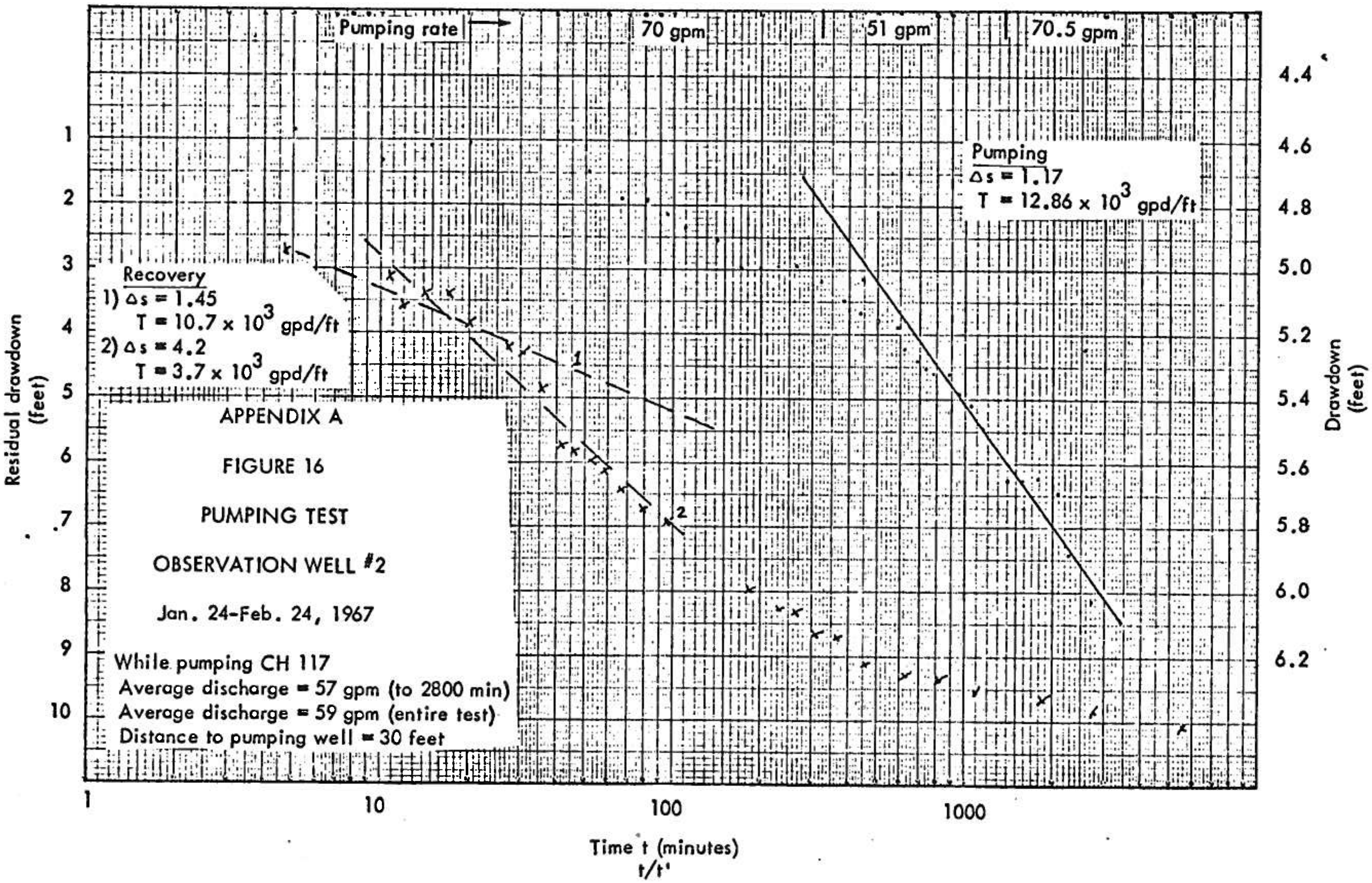
FIGURE 15

PUMPING TEST

OBSERVATION WELL #1

Jan. 24-Feb. 24, 1967

Average discharge = 57 gpm (to 2800 min)
Average discharge = 59 gpm (entire test)
Distance to pumping well = 12 feet



APPENDIX A

FIGURE 16

PUMPING TEST

OBSERVATION WELL #2

Jan. 24-Feb. 24, 1967

While pumping CH 117
Average discharge = 57 gpm (to 2800 min)
Average discharge = 59 gpm (entire test)
Distance to pumping well = 30 feet

APPENDIX B

SPECIFICATIONS FOR DEWATERING WELLS

This section gives the general specifications for all wells which are to be pumped to dewater portions of the lease.

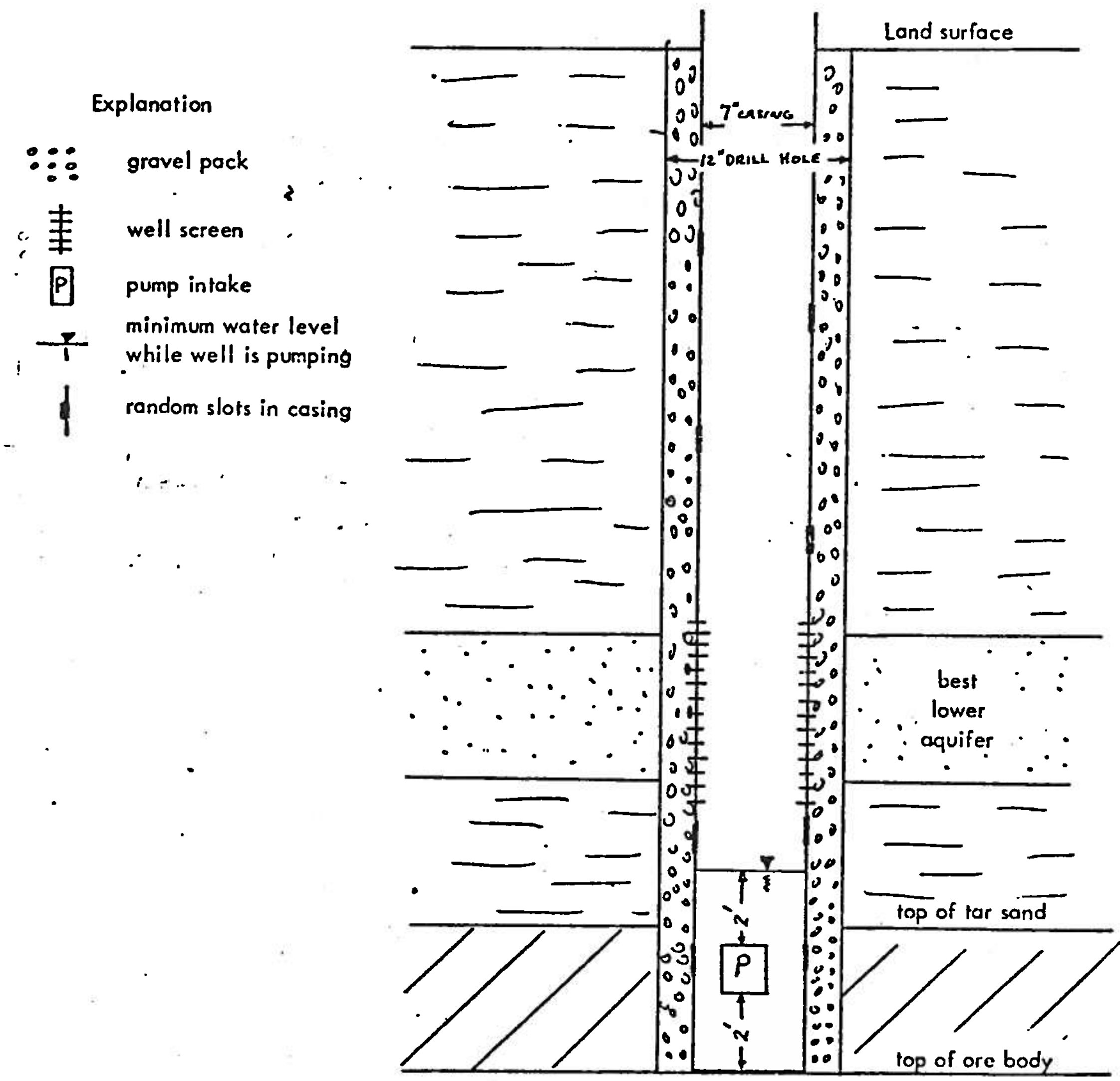
Due to the fine-grained nature of many of the units in the subsurface it is recommended that these wells be completed with 7 inch casing in a 12 inch hole which has been gravel-packed from the top of the ore body to the surface. Figure 1 is a schematic diagram of any dewatering well. These wells should be extensively developed to insure that sediment will not enter the well and cause wear on the pump. The casing should be slotted with one 6-inch long opening every foot for the entire distance from the surface to the top of the ore body. This will permit water to enter the well from all depths. In addition, it is recommended that a 5-foot segment of well screen be placed across the best aquifer in the lower half of the hole.

It is recommended that each dewatering well be pumped initially at as high a rate as possible for the type of pump selected. The water level in the pumping well should be monitored so that the discharge rate can be reduced when the level comes within 2 feet of the pump intake. This monitoring can be done either manually or with some sort of self-regulating device which would adjust pumping rates to maintain the minimum 2 feet of head above the pump intake. The self-regulating system would require only occasional supervision while the manually adjusted system would necessitate much closer observation. Pumps are usually damaged extensively if run in a dry state, thus any monitoring system must allow for sufficiently frequent observations to prevent this situation.

It is expected that sustained pumping rates between 10 and 100 gpm will be possible in the lease. This should be the range of capacity of whatever type of pump is selected for use in the dewatering wells.

APPENDIX B

Schematic Diagram of Dewatering Well



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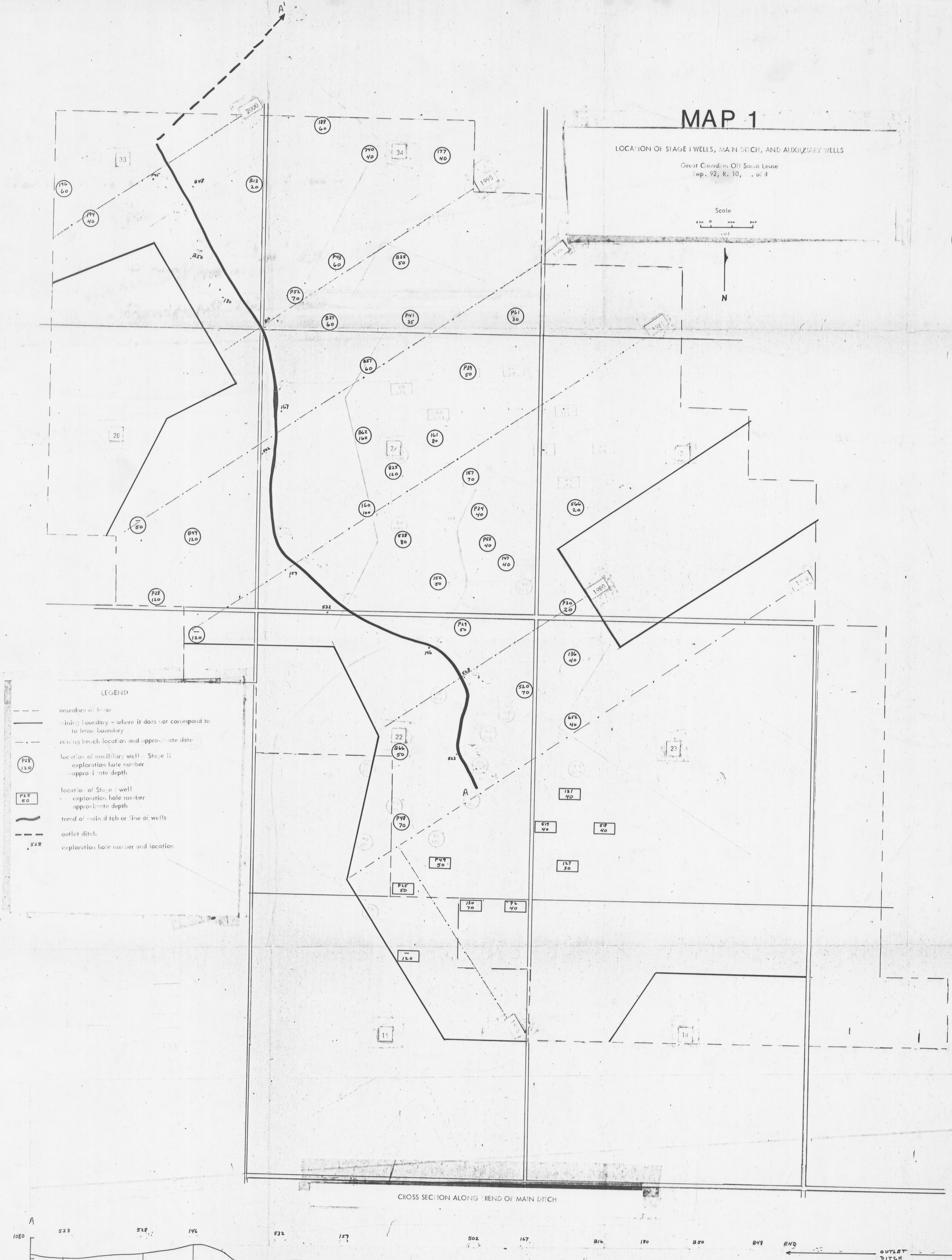
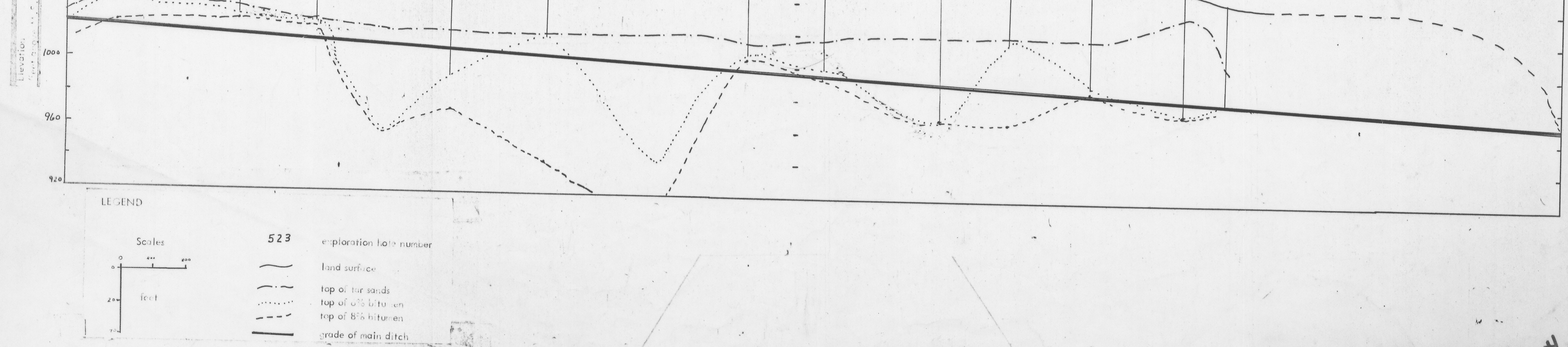


CROSS SECTION ALONG TREND OF MAIN DITCH



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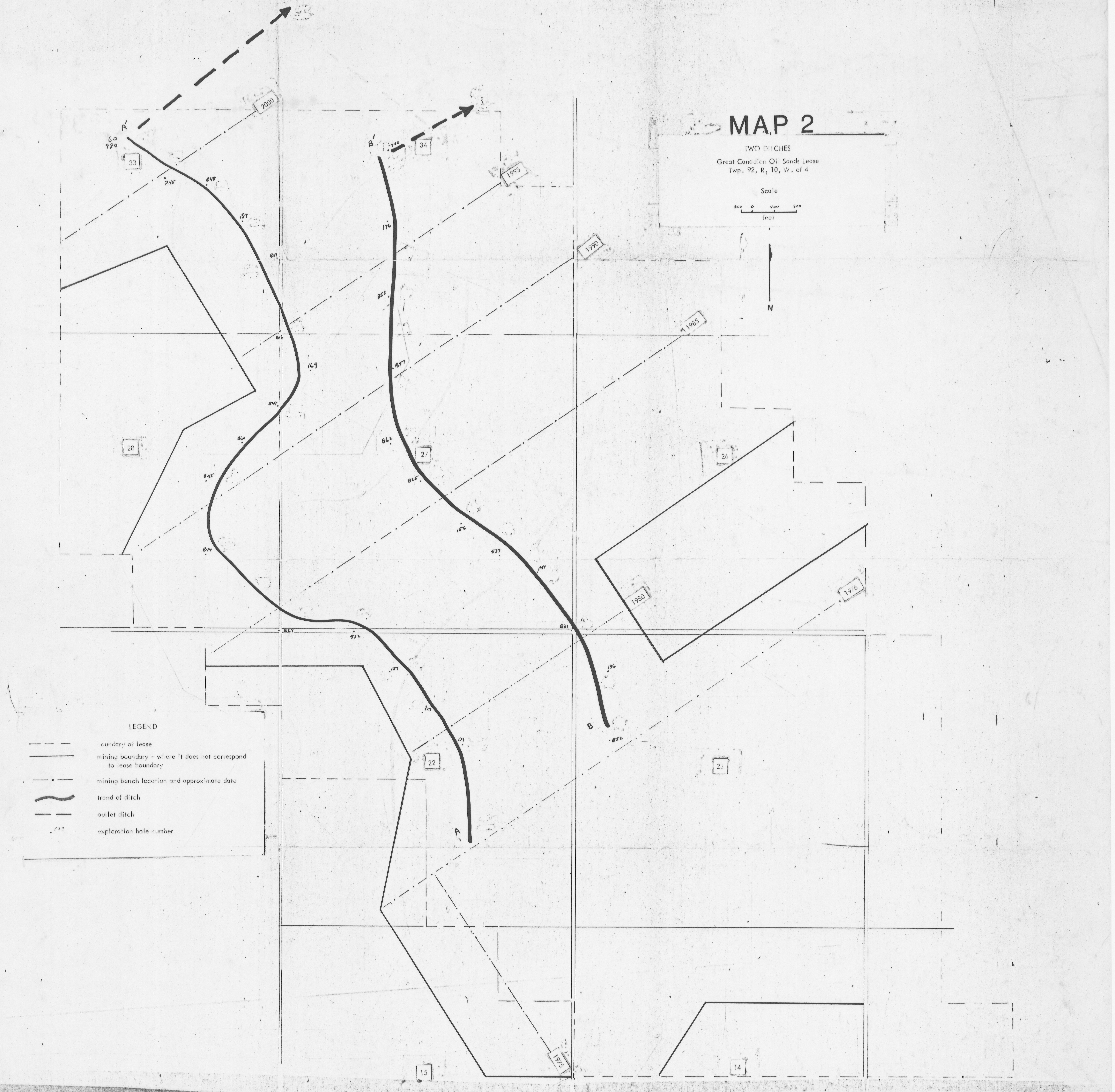

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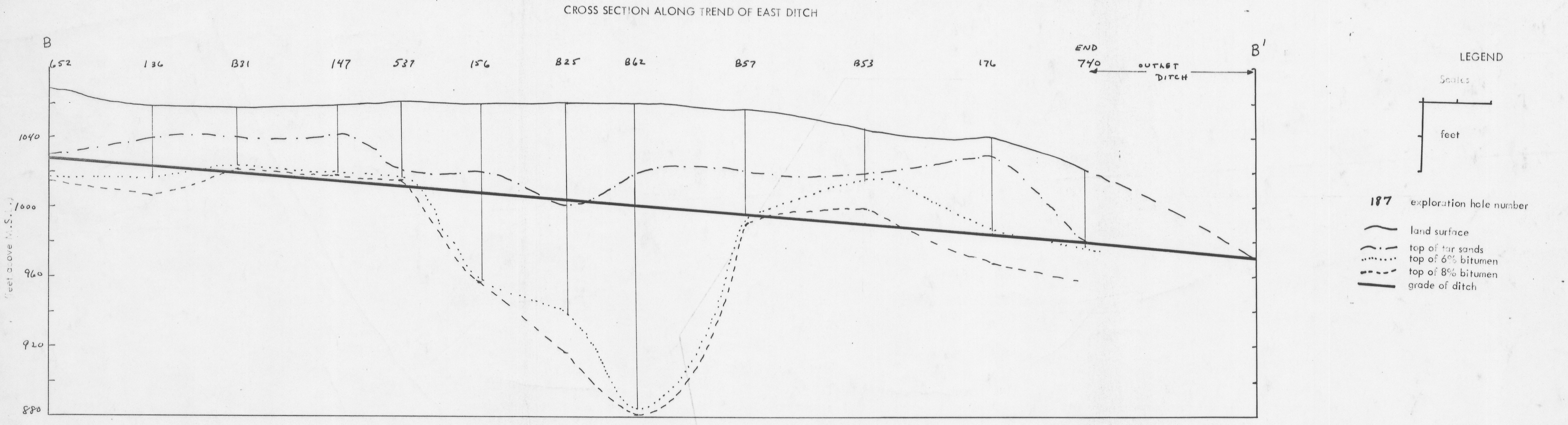

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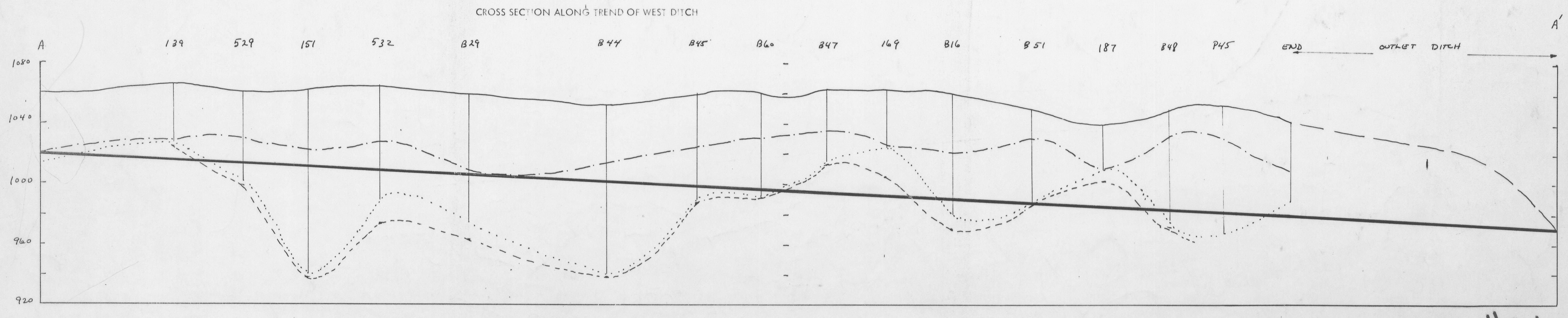

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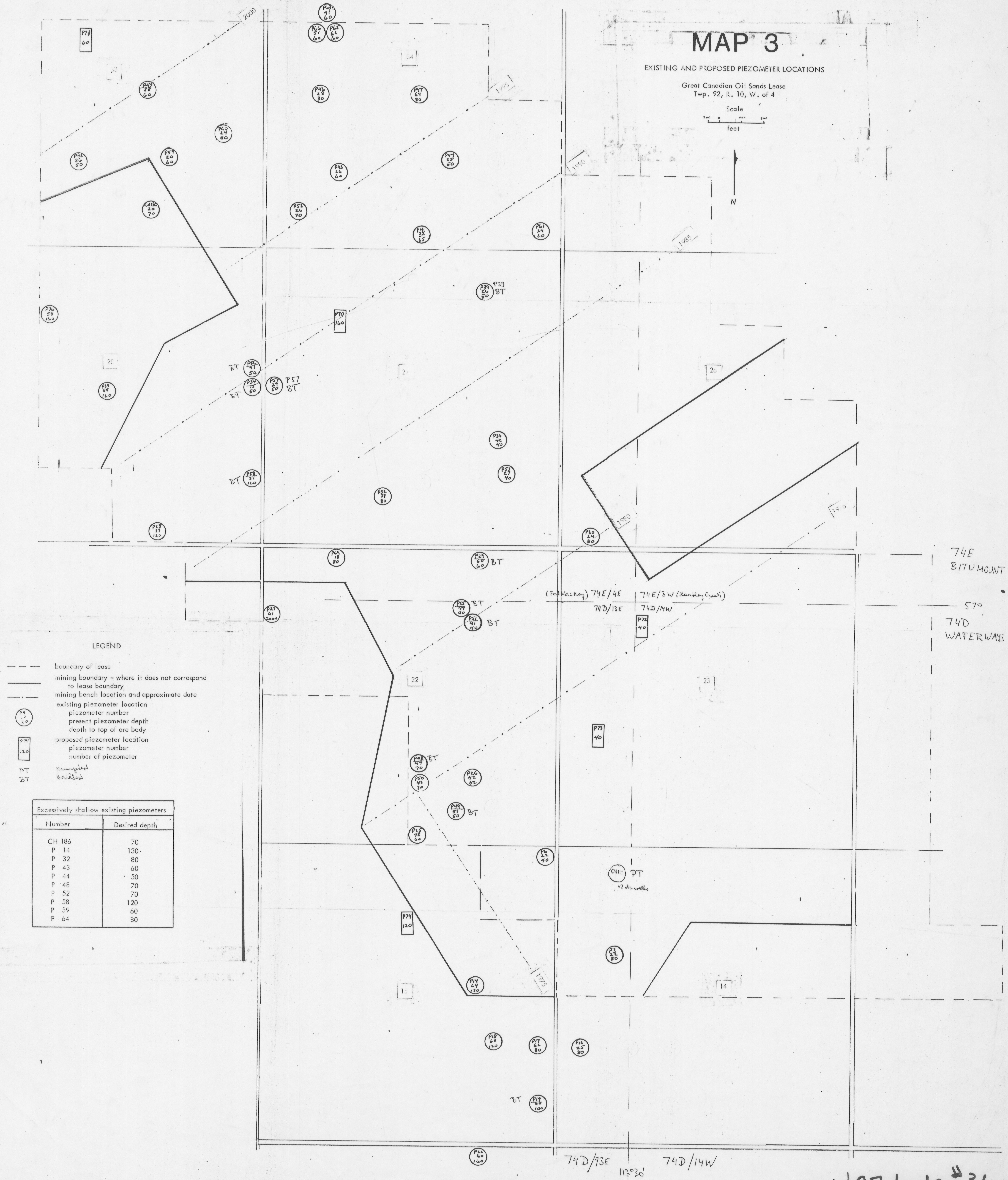

Excessively shallow existing piezometers	
Number	Desired depth
CH 186	70
P 14	130
P 32	80
P 43	60
P 44	50
P 48	70
P 52	70
P 58	120
P 59	60
P 64	80

1971-10 #3/3